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DEPARTMENT OF THE ARMY TECHNICAL MANUAL

DRAINAGE STRUCTURES, SUBGRADES, AND BASE COURSES

HEADQUARTERS, DEPARTMENT OF THE ARMY
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HEADQUARTERS
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DRAINAGE STRUCTURES, SUBGRADES, AND BASE COURSES

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*This manual supersedes chapters 1, 2, 3, 5, 9, 16, and 24 of TM 5-250, 7 August 1957.

CHAPTER 1

INTRODUCTION

1. Purpose

This manual is for engineer officers and technically trained noncommissioned officers responsible for the construction of drainage facilities (including the hydraulic design of culverts) and the preparation of subgrades and base courses. Particular emphasis is placed on theater-of-operations construction.

2. Scope

a. This manual covers pertinent drainage, subgrade, and base course terminology, the construction of drainage facilities, and the preparation of subgrades and base courses.

b. The information contained herein is applicable without modification to both nuclear and nonnuclear warfare.

c. Users of this manual are encouraged to submit comments or recommendations for changes to improve the manual. Comments should be keyed to the specific page, paragraph, and line of the text in which the change is recommended. Reasons should be provided for each comment to insure understanding and proper evaluation. Comments should be forwarded direct to the Commandant, U.S. Army Engineer School, Fort Belvoir, Va.

CHAPTER 2

THEATER-OF-OPERATIONS CONSIDERATIONS

3. Characteristics of Theater-of-Operations Construction

Forward areas are constantly exposed to shelling and all areas are subject to bombing. The resulting dust, mud, craters, and blocked drainage interfere with operations and sometimes cause confusion. Under these conditions, which must be considered normal in a theater of operations, engineers must provide and replace facilities. At the front, the exigencies of the military situation usually necessitate rough, hasty work designed primarily to meet immediate needs. Engineers must also provide some deliberate rear-area construction, but little construction of the permanency represented by usual civilian practice is expected of engineers in a theater of operations.

4. Theater-of-Operations Construction Directives

Based on the tactical or strategic situation, higher authority dictates certain requirements or specifications when assigning construction missions. These include—

a. Time Allotted for Completion. In forward construction, speed is usually the dominating factor.

b. Type of Construction. Examples include main supply route, fair-weather road for temporary traffic relief, forward tactical airfield, or heavy bomber airfield.

c. Location. Specific location should always be determined by a competent, trained engineer.

5. Basic Considerations

The following factors must be considered in all drainage, subgrade, and base course work and are of particular importance in a theater of operations:

a. Economy of Time. The nearer the operation is to the front, the more vital the time element becomes. Time is saved by efficient use of manpower, power equipment, handtools, materials, and other facilities available.

b. Simplicity. Simple designs requiring a minimum of skilled labor and utilizing available materials and supplies should be used.

c. Economy of Materials. Facilities provided by construction projects should be held to a minimum, with minimum requirements governed by the purpose of the completed project. Materials must be conserved, particularly those shipped from the zone of the interior. Local materials should be used whenever practicable.

d. Planning and Management. Good planning, careful scheduling, and thorough supervision constitute effective job management. Such management hastens job completion and economizes on time, labor, equipment, and materials. Whenever possible, stage construction should be used to permit early use of the facility while further construction and improvement continue.

e. Air Defense Measures and Camouflage. Aerial attack on vital installations must be expected. The likelihood and effectiveness of such attacks, however, are often minimized by the selection of a site that gives protective concealment, and by the use of antiaircraft weapons and camouflage.

CHAPTER 3
DRAINAGE CONSIDERATIONS

6. Importance of Drainage

a. Drainage is an important consideration in the planning, design, and construction of military roads and airfields, and it remains important during their construction and use. The entire serviceability of a road depends on the adequacy of the drainage system. The washout of a single culvert may close a road to traffic at a vital time. The development of a soft spot may lead to rutting, displacement, and eventual closing of a road for repairs. Properly designed and constructed drainage systems are equally vital to the functioning of an airfield. It is most important that adequate drainage facilities be provided, to remove effectively any surface water from the runways, taxiways, and hardstands. Also, all runoff from adjacent areas must be intercepted, to prevent possible damage to the pavement. One severe accident resulting from inadequate drainage may offset any difference between the cost of reasonably adequate and the cost of less than adequate facilities.

b. In permanent construction, subsurface drains are frequently used because property values will not permit large surface ditches, particulary for disposal of collected runoff. In contrast, design in a theater of operations uses surface ditching almost exclusively, because of the logistical limitation on pipe and the absence of storm-sewer systems into which to discharge collected runoff. The comparative ease of repair and cleaning of ditches makes them preferable to pipe for subsurface drainage in a theater of operations.

c. Adequate drainage, including the use of pumping facilities of even a temporary nature, is of prime importance during the construction period. Inadequate drainage results in much need for repair and maintenance of construction equipment and seriously affects its efficient operation. The first construction work on any project should provide drainage for the work to follow. As construction progresses, every effort should be made to complete the drainage system as planned, in order to avoid any damage which might be caused by accumulations of mud, water, ice, and debris. Flooding caused by inadequate drainage may lead to failure of road and airfield surfaces.

d. Engineer officers and noncommissioned officers should be fully aware of the function of drainage, including adequate drainage during construction, and the proper methods of providing it. The importance of drainage in road and airfield design and construction cannot be overemphasized.

7. Types and Functions of Drainage

All drainage can be classified as one of two types: surface or subsurface. Classification depends on whether the water is on, or below, the surface of the ground at the point where it is first intercepted or collected for disposal.

a. Surface Drainage. Surface drainage provides for the collection and removal of water from the surface of roads, runways, taxiways, and hardstands. This is important because water on the surface interferes with traffic, may cause erosion, and, if allowed to infiltrate, causes injury to the subgrade. Water on runway surfaces increases the hazard of landing and takeoff operation and can cause icing in cold weather. Surface drainage also provides for the interception, collection, and removal of surface water flowing toward road and airfield surfaces from adjacent areas.

b. Subsurface Drainage. Subsurface drainage is similar in some respects to surface drainage. Impervious strata may form well-defined channels and reservoirs for subsurface or ground water. Water is present under the surface because of infiltration of surface water and ground water. Surface water seeps down through open or unsealed surfaces, or laterally along the top of impervious soil or rock layers. Ground water may pond above impervious strata to form a subsurface lake or perched water table. Properly designed and maintained surface drainage systems should reduce the need for special facilities for control and disposal of ground water. Subsurface drainage is provided to intercept, collect, and remove from them, any flow of ground water into the base course or subgrade; to lower high water tables; to drain water pockets or perched water tables; or for any combination of these purposes.

8. Drainage Characteristics of Soils

a. Soil symbols such as GW, SP, ML, CH, Pt, etc., are discussed in detail in paragraph 33. The general drainage characteristics of soils are given in column 11 figure 61. Additional information about the movement of water through soils is contained in TM 5–541, including complete discussions of capillarity and permeability.

b. Soils may be divided into three general groups on the basis of their drainage characteristics, as follows:

(1) *Well-draining or free-draining soils.* Clean sands and gravels, such as those which are included in the GW, GP, SW, or SP groups, fall into this classification. These soils may be drained readily by gravity systems. In road and airfield construction, for example, open ditches

sometimes may be used in these soils to intercept and carry away water which comes in from surrounding areas. This approach is very effective when used in combination with the sealing of the surface to reduce infiltration into the base or subgrade. In general, if the ground-water table around the site of a construction project is controlled in these soils, then it will also be controlled under the site.

(2) *Poorly draining soils.* In this group are included inorganic and organic fine sands and silts, and organic clays of low compressibility, together with the course-grained soils which contain an excess of nonplastic fines. Soils in the ML, OL, MH, GM, and SM groups, and many of the Pt group, might generally fall into this category. Drainage by gravity alone is apt to be quite difficult for these soils. Subsurface drainage systems may sometimes be effective, if the water table can be lowered below the effective height of capillary rise.

(3) *Impervious soils.* Fine-grained, homogeneous, plastic soils and coarse-grained soils which contain plastic fines belong in this category. This would normally include the GC, SC, CL, CH, and OH groups. Subsurface drainage is so slow in these soils that it is of little value in improving their condition. Any drainage process is apt to be difficult and expensive.

9. Temporary Drainage During Construction

Proper attention to drainage during the construction period will frequently eliminate costly initial delays and future failures due to saturated subgrades. A careful consideration of the following items will aid in maintaining satisfactory drainage during construction.

a. Diversion and Outfall Ditches. Drainage measures are necessary at the site to eliminate water which would interfere with construction operations. These measures include excavating diversion ditches to concentrate all surface water in natural channels, and building outfall ditches to drain low or swampy spots. Such work is an initial operation and may proceed at the same time as clearing and grubbing. Careful consideration should also be given to the drainage of all construction roads, equipment areas, borrow pits, and waste areas.

b. Use of Existing Ditches and Drainage Features. Maximum use should be made of existing ditches and drainage features. Where possible, the grading operations should proceed downhill, both for economical grading and to utilize natural drainage to

the greatest extent. Backfilling of existing ditches and drainage channels should be be so scheduled as to permit the longest possible use of these structures for temporary drainage.

c. Use of Temporary Crowning. It is necessary to maintain well-drained subgrades and base courses at every stage of construction to prevent harmful saturation. When work is temporarily suspended, a particular effort should be made to remove runoff from the site. Cut and fill sections must be crowned and high shoulders must be lowered.

d. Coordination of Drainage Plans. Construction drainage plans should be coordinated with the layout and design of final drainage facilities to insure maximum use of temporary drains in constructing permanent facilities.

10. Drainage Maintenance

The drainage system must be maintained so that it can function efficiently at all times. This goal can be obtained through adequate maintenance inspections of the structures, with careful attention to removal of debris and prevention of erosion. Periods of dry weather must be utilized in improving the drainage system and correcting and preventing drainage failures. Any deficiencies in the original drainage system's layout must be corrected as they are discovered.

a. Surface Drainage. Defective or inadequate drainage is responsible for many pavement failures and much deterioration. Ponding or delayed runoff of surface water, although sometimes incorporated as a deliberate feature of economical drainage design, will cause seepage unless the soil in contact with the water is impervious or protected by a waterproof layer. Areas should be marked where inspection after rains reveals ponding of surface water. Correction is then made by filling or raising local depressions, by providing additional culvert capacity, and by providing outlets for water obstructed by higher shoulders. Penetration of surface runoff through pavement is controlled by sealing joints and cracks.

b. Ditches and Drains. Drainage ditches must be kept clear of weeds, brush, sediment, and other accumulations of debris that obstruct the flow of water. Ditches are maintained as to line and grade, and sags and minor washouts are corrected as they occur. Unnecessary blading or cutting, which destroys natural ground cover, is avoided in cleaning and shaping. Dense sod is developed to stabilize open ditches. Where vegetation is not effective because of soil or moisture conditions, erosion may be corrected by lining the ditch with riprap or compacted bituminous premix. Checkdams in side ditches should be inspected and cleaned regularly.

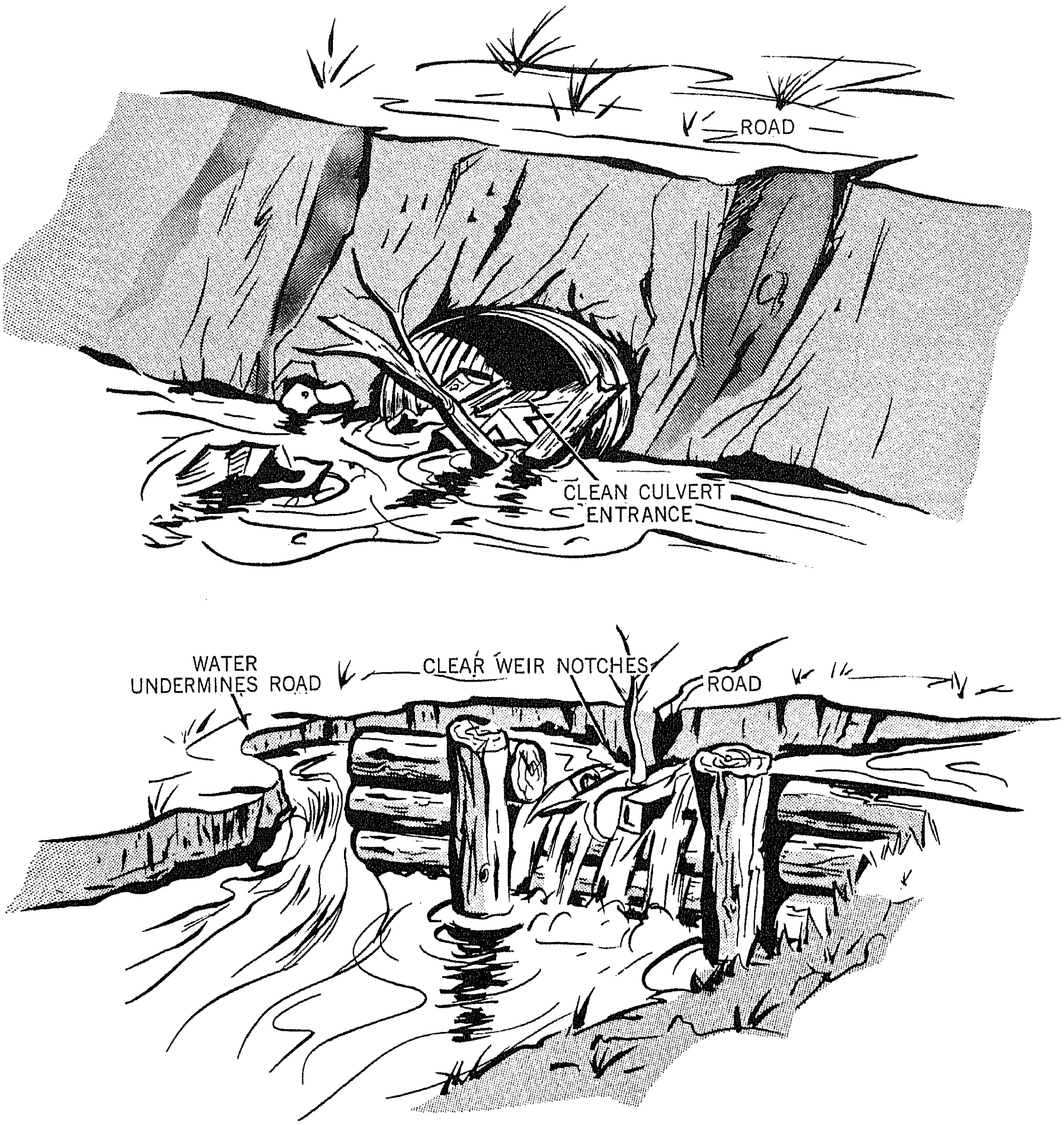


Figure 1. Poor drainage maintenance. Culvert and checkdam clogged with debris.

The weir notch of a checkdam must be kept clean or the water will cut into the road at the edge of the dam (fig. 1). The aprons of checkdams must also be maintained, and paving material replaced when washed out. Protecting dikes or berms may be re-

quired along the top of high-fill slopes to prevent gullies and washes.

c. Culverts. Culverts must be kept clear of debris and sediment (fig. 1), and water prevented from cutting around or undermining them. Frequent inspection is necessary to determine whether culverts are fuctioning properly. Cleaning is usually necessary after heavy rains, and is normally done by hand.

d. Shoulders. Shoulders shoud be kept smooth and graded so that they drain water away from the road toward the ditch. On a paved road new material should be hauled in to replace any shoulder material eroded away. Material cleaned out of ditches can often be used to rebuild shoulders. Shoulders should be kept bladed flush to the edges of the pavement to prevent water seepage into the subgrade. If, for any reason, this is not feasible, outlets should be provided for water obstructed by high shoulders. Stone-filled French drains, normal or at a down-grade angle to road or runway, also aid runoff where heavy precipitation is a seasonal occurrence. Such drains help minimize rutting or washout of shoulders and consequent weakening of road edges, thus reducing later repair requirements for materials, labor, and time.

e. Winter Maintenance. If there are accumulations of snow, special attention must be given to drainage maintenance during thaws. Side ditches are cleared of snow, and channels are opened through snow accumulations on the shoulders to permit water to escape into the ditches. Every precaution is taken to prevent melt water from ponding on the runway or road surface, on the shoulders, or in side ditches. Culverts and drains are kept free of ice and snow.

CHAPTER 4

DRAINAGE STRUCTURES

11. Surface-Drainage Structures

Surface water is removed from road and airfield pavements or wearing courses by provision of adequate crown or transverse slopes. Drainage systems must accommodate this water and the surface water from areas adjacent to the road or airfield pavement, as well as that which flows in from outlying portions of the drainage area. Natural and artificial means are used, generally in combination, to collect, intercept, control, and dispose of surface water. Natural elements include streams, lakes, dry runs, and ponding areas. Artificial or manmade facilities include open channels or ditches, storm drains, culverts, bridges, fords, dips, and the necessary auxiliary structures. Underground pipelines with inlets (storm drains) are seldom justified in theater-of-operations construction. Some of the structures used in surface drainage are shown in figure 2.

a. Open Channels. Open channels or ditches, alone or in combination with natural watercourses, provide the simplest as well as the cheapest and most efficient method of handling surface water. Gutters, or combination curbs and gutters, are used to collect and control surface runoff where open ditches would be hazardous or impractical. They are most commonly employed along streets in densely settled areas, and along roads through heavy cuts where the width of the roadway is limited.

(1) *Side slopes.* The cross section of open ditches or channels is kept reasonably constant for convenience in construction and maintenance. Slopes of the order of 4 to 1 can be constructed by equipment and are usually built in less time and maintained more easily than are ditches constructed using manual labor only. Back-slopes are normally a continuation of the cut-slope ratio. At every opportunity, drainage water should be diverted or discharged from collecting channels into natural drains that will carry it away from the site, thereby reducing seepage into the subgrade.

(2) *Ditch grades.* Open ditches, with channels kept clear, function satisfactorily with comparatively flat grades. However, the gradient is governed by the type and condition of the soil, together with the configuration of the ground. The minimum grade should not be less than 0.5 percent, and to assure positive runoff, a 1.0 percent

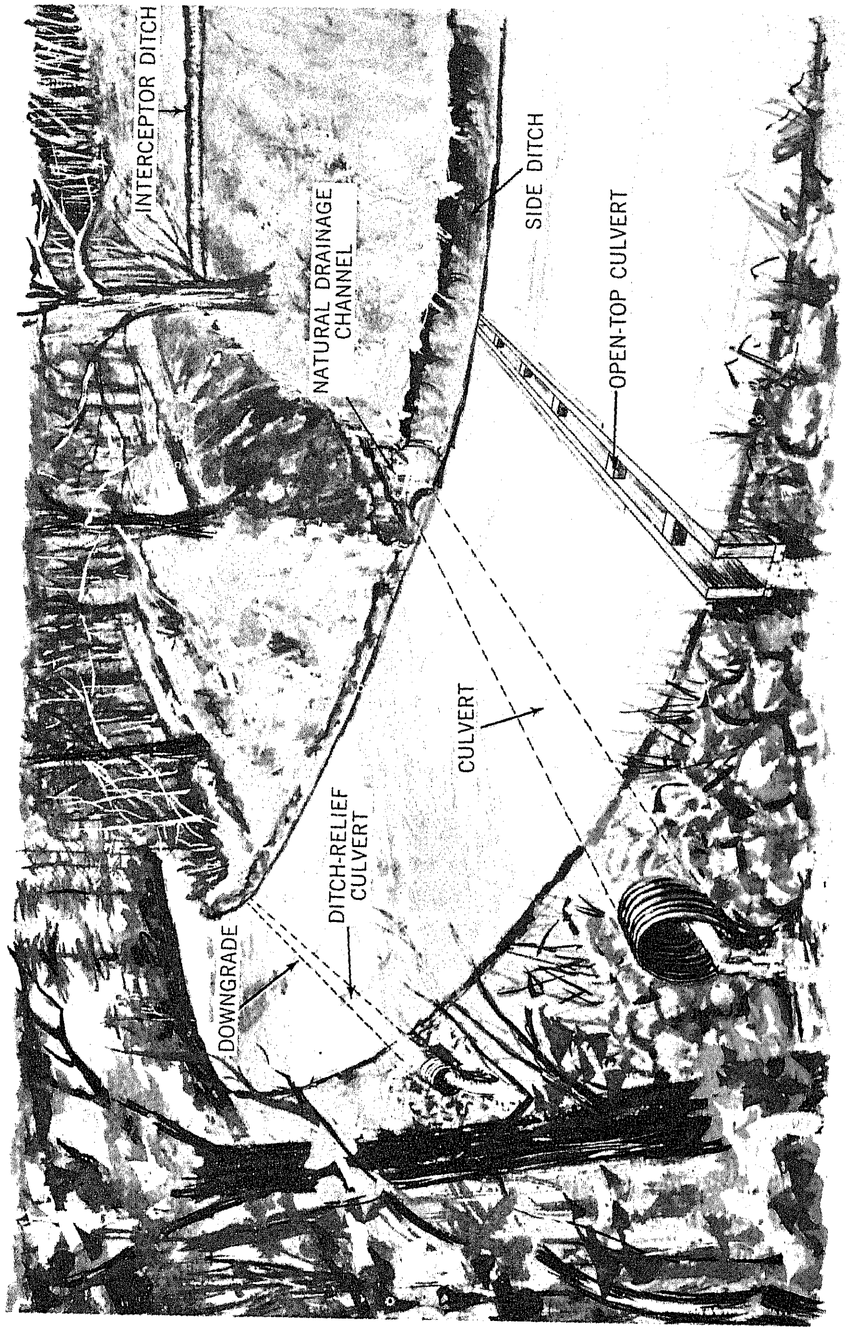


Figure 2. Surface-drainage structures.

minimum grade is preferable. In ground subject to erosion at the normal or governing slope of the ground, the gradient can be flattened and erosion arrested through the use of checks at convenient intervals. For further discussion of erosion control see paragraph 13.

(3) *Side ditches.* Side ditches are a potential danger to traffic. To reduce this danger to a minimum, the shoulders should be constructed as wide as practicable and with a uniform slope toward the ditch. If possible, the required cross-sectional area should be obtained by constructing a broad, shallow trapezoidal ditch. This type of ditch gives great capacity. Its minimum depth below the edge of the shoulder should be 1½ feet. The minimum practicable trapezoidal section is 1½ feet deep and 2 feet wide at the bottom. Side slopes not steeper than 1½ to 1 in cohesive soil, and 3 to 1 in sandy or loamy soils, should be used. When possible, ditches should be deep enough to lower the ground-water table, if high, to below the subgrade elevation, and thus permit drainage of the subgrade by seepage into the ditch. Where it is economically possible to develop and maintain turfed side slopes, the side slopes of ditches should not be steeper than 4 horizontal to 1 vertical, to facilitate mowing and operations incident to maintenance. Ditches with comparatively flat side slopes must be protected by rigid traffic control against indiscriminate crossing of these ditches by vehicles. The longitudinal grade of the ditch must be great enough to provide free flow of water along the ditch, with velocities that are self-cleaning but do not cause erosion. To prevent the flow from standing in low places and seeping into the ground, a minimum grade of 0.5 percent should be used. It is impossible to set a maximum grade because of the tendency of soil to erode. The maximum grade, therefore, depends upon the nature of the soil. Soils with a high percentage of rock erode more slowly than clay or sandy soils. Where longitudinal grades of 4 percent or more are encountered, high velocities, induced by large volumes of water, may require paved ditches, baffles or drop walls, or paved spillways to control erosion.

(4) *Interceptor ditches.* Much of the surface water originating on adjacent areas which slope toward a construction site can be intercepted and carried off in interceptor ditches before it reaches the site. Intercepting the surface water reduces the volume of flow into drainage

facilities within the operating areas and prevents drainage structures from reaching uneconomical sizes. Without interceptor ditches, all of the water from a hill would flow along the inside or cut-side ditch. The pipe necessary to provide for the gradual increase of flow would be excessive in size and very expensive.

(5) *Placing of ditches along fills.* In some cases, drainage ditches must be constructed parallel to the edge of a fill. When this is necessary, the ditch should be dug into the natural ground. The fill-ground-line intersection should not be used as the bottom of the ditch because surface water would tend to infiltrate and weaken the fill. A berm should be placed between the toe of the fill and the ditch when the fill is over 4 feet high. Figure 3 illustrates both the correct and the incorrect method for locating a ditch parallel to a fill.

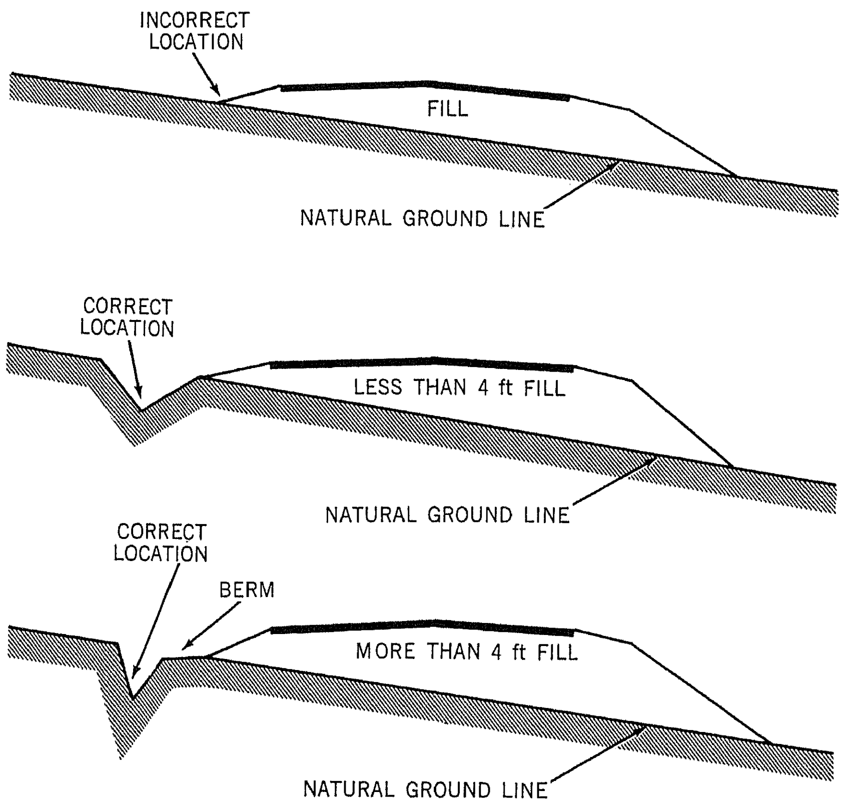


Figure 3. Construction of ditches parallel to fills.

b. Dikes. Dikes, berms, or intercepting embankments are used along shoulders on high fills, or along the tops of cut slopes, to collect runoff. Such runoff may be directed into ditches or natural drainage courses to prevent the erosion of unstable slopes. Flumes are installed on the surface of steep slopes to carry accumulations of surface water to open ditches or natural drainage channels.

c. Storm Drains. Where space is restricted, or the natural slope of the ground is unsuited for drainage by open ditches or channels, it may be necessary to install storm drains or storm sewers to provide for the disposal of surface water. Handling water by surface elements is more efficient and economical than doing so by underground structures, and, as indicated in paragraph 6*b*, the installations of storm drains is seldom justified in theater-of-operations construction. A storm-drainage system consists of an underground layout of glazed or unglazed vitrified clay, reinforced or nonreinforced concrete, asbestos-cement, bituminized fiber, or metal pipe, or box- or arch-shaped structures of concrete, metal, or masonry, with the necessary inlets and manholes. A storm-drainage system is not like a sanitary layout, which must gather all sewage into one collecting system and carry it to a treatment or disposal plant. Storm drains are best installed in small units, each unit draining a small area and carrying the water to the nearest watercourse.

d. Culverts. Culverts are used under roads, taxiways, and occasionally, under runways, to carry water that cannot be diverted economically to natural drainage channels by other means. Culverts differ from storm drains in that they are relatively short and conform generally to the grade and alinement of the open ditch, stream, or natural drainage course at inlet and outlet ends. Where conditions warrant, headwalls or wingwalls of concrete, masonry, or timber construction are used. For additional information about culverts and culvert hydraulic design, see paragraphs 14 through 21.

e. Bridges.

(1) Bridges, like fords and dips, are drainage structures since they provide a means of crossing natural or artificial drainage channels which, otherwise, would require fills and culverts. The design and construction of bridges are discussed in TM 5–260.

(2) If water is shallow, expedient underwater bridges can be constructed from boulders, gravel, and earth- or sand-filled burlap or rice bags. Underwater bridges have the advantage of being difficult to see and also to destroy from the air.

f. Fords. A ford is a shallow place in a stream where the bottom permits the passage of personnel and vehicles.

(1) *Use.* Fords are used when limitations of time, the tactical situation, configuration of the ground, or lack of suitable structural materials make their use practical or necessary. Fords are unreliable, because of increases in depth in flash floods and deterioration under heavy traffic. Streams in mountainous and desert areas are subject to freshets, and the bottoms are frequently covered with large rocks, which make wheeled traffic difficult. In level country, the bottom is likely to be either mud or quicksand. Gravel is the best bottom.

(2) *Requirements.* Fords are most often found in reaches of streams between pronounced bends. Characteristics of a good ford are a slow current, usually less than 2 miles per hour, not subject to sudden freshets; low sloping banks with good approaches, preferably gravel and sand, not marshy; a uniformly increasing depth; and even, hard, and tenacious bottom. Minimum requirements for military fords are given in table I.

Table I. Minimum Requirements for Military Fords

	Maximum depth (feet)	Minimum width (feet) (one-way traffic)	Type bottom	Maximum allowable slope on approaches*
Infantry	3½	3 (single file)	Firm enough to prevent sinking.	1:1
		7 (column of 3's)	Boulders and obstacles removed.	
Trucks	2	10	Firm and smooth	3:1
Light tanks	1–3	10	Firm and smooth	2:1
Medium tanks	2–4	10	Firm and smooth	2:1
Heavy tanks	4–6	12	Firm and smooth	2:1

*Based on hard dry surface. If wet and slippery, slope must be less.

(3) *Construction.* If approaches are too steep, the banks must be leveled off. Material should be placed off to the sides on the banks, and not in the stream where it will make a dam and obstruct the crossing. Some type of surfacing is desirable for a short distance on the approaches so they do not become slippery from water dripping off vehicles. Streams not fordable due to short gaps in which the water is too deep can be made fordable by filling the gaps with rock and coarse gravel. Fords in sluggish or muddy streams may be improved by cover-

ing the bottom with timber, pierced-steel planks, willow mattresses, brush, flat rock, or coarse gravel.

g. Dips. Dips are paved fords, used for crossings of wide, shallow arroyos or washes in semiarid regions subject to flash floods and in other locations where the construction of a bridge is impractical. The pavement should be protected on its upstream side by a cutoff wall, and on its downstream side by an apron. An apron may also be provided on the upstream side to prevent erosion. The pavement may be macadam, concrete, or timber. The cutoff wall should extend 18 to 24 inches below the paved surface. Riprapping, rubble masonry, concrete, timbers, or logs may be used for the surfacing of the aprons. Protection should be carried well away from the roadway edges. In some cases culverts are installed underneath the dip to take the normal flow of the stream. In areas where the stream channel is normally dry, the dip should be constructed about 6 inches below the bed of the channel. This procedure will minimize scour, and the soil deposited during flood time can be easily removed. Alinement should be straight and the pavement location should be shown by two marking posts at each end and by as many intermediate posts as necessary. One or more of the posts should be gaged to indicate the depth of water during floods. Dips should be sloped toward the downstream edge of the pavement in lieu of providing a center crown.

12. Subsurface-Drainage Structures

a. Effect of Excess Water. In most cases the presence of excess water in the subgrade or base course, whether due to surface penetration, ground-water seepage, or capillary action, reduces stability and load-bearing capacity and adds to the danger of frost action.

b. Control of Subsurface Water. The drainage or control of subsurface water may be accomplished by a system of subsurface drain pipes placed in trenches with a backfill of suitable filter material. An alternative method is the use of a system of open channels or ditches which may be new channels or improved existing channels. In addition, there are special methods for subsurface drainage, such as vertical or dry wells. These are designed to drain a perched water table into a lower ground-water reservoir. Subsurface drains may be divided, in accordance with their purpose, into base, subgrade, and intercepting drainage. Base drainage consists of removing water from the base course beneath a pavement. Subgrade drainage consists of removing water from the subgrade beneath a pavement. Intercepting drainage collects and removes water flowing in previous strata, or from springs, before it can enter the subgrade. In some situations ground water may be controlled by thickening the base course.

(1) *Thickening base course.* This consists of increasing the depth of the base course so that the top of the base course is high enough above the natural water table to avoid trouble. With base-course material that is reasonably free from capillary action, the usual required specification is that the ground-water level be at least 4 feet below the top of the finished base course. To prevent frost action will require a minimum of 5 feet. This solution is indicated when a gravity drainage system is not feasible, the condition is a local one such as a swamp crossing, and there is a sufficient supply of suitable base-course material for the fill. If suitable material is not available, corduroy construction may be used.

(2) *Open channels (ditches) as subsurface drains.* Another means of subsurface drainage is by deep V-ditches. These ditches are easily built, are readily enlarged, and provide positive interception of subsurface water before it reaches the area being protected. Also, if the water level in natural drainage channels can be lowered, it may be possible to lower the ground-water level of the surrounding area, particularly if the surrounding soils are pervious. However, such ditches (or natural channels) are often a traffic hazard, and they are also subject to erosion. In those cases where right-of-way problems, traffic situations, and erosion difficulties make the use of open ditches impractical, it may be necessary to use a subsurface drainage system consisting of blind or "French" drains, or of perforated or open-joint piping as discussed below.

(3) *Base drainage.* Base drainage is required not only where frost action occurs in the subgrade, but also where water destroys the built-in stability of the base. Base drainage generally consists of subsurface drain pipes laid parallel and adjacent to pavement edges with pervious material joining the base and the drain. (Figure 4 shows a typical section of a base-drain installation.) Base drainage is required where frost action occurs in the subgrade beneath the pavement and where ground water rises to the bottom of the base course as a result of natural conditions or from ponding of surface runoff. At locations where the pavement may become temporarily inundated and there is little possibility of the water draining from the base into subgrade, base drainage will be required. Table II establishes the criteria to follow in these cases. Base drainage is also required at the low point of longi-

tudinal grades in excess of 2 percent where the subgrade coefficient of permeability is less than 1 x 10^{-3} ft/min. The coefficient of permeability is a property of the soil and is defined as the discharge velocity at a unit hydraulic gradient. This value, based on Darcy's equation,

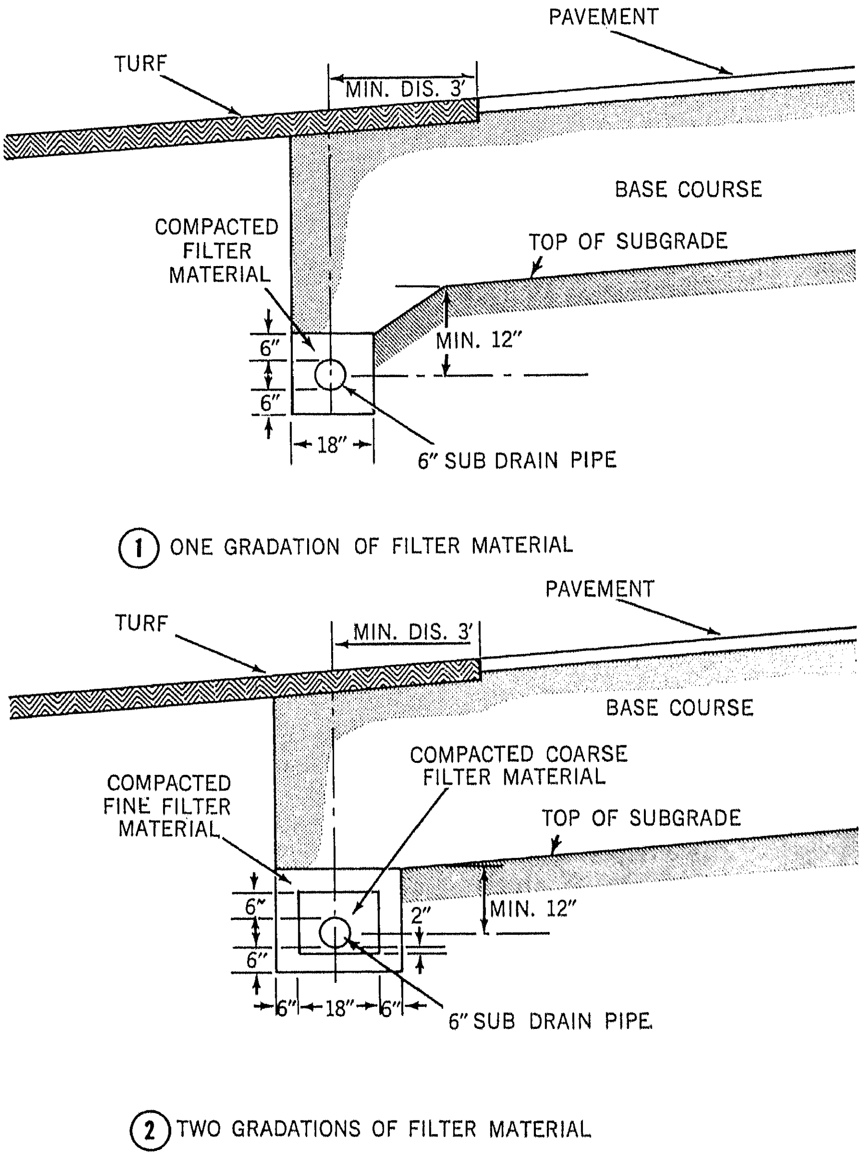


Figure 4. Typical details of a base-drain installation.

is determined experimentally, either by laboratory test or by an actual field test of the soil involved. It is expressed in units of velocity such as feet per minute (ft/min) or centimeters per second (cm/sec) and varies from 197×10^{-4} ft/min for gravel and gravel-sand mixtures to 197×10^{-10} ft/min for silts and clays. Coefficients for various soil classifications may be found in TM 5-541.

Table II. Base Drainage Required if Subgrade Coefficient of Permeability Is Smaller Than Stated Feet Per Minute and Inundation May Occur

Depth to ground water (ft)	Coefficient of permeability less than:
Less than 8	1×10^{-5} ft/min
From 8 to 25	1×10^{-6} ft/min
Over 25	1×10^{-7} ft/min

(4) *Subgrade drainage.* Subgrade drainage is required at locations where seasonal fluctuations may be expected to cause the ground-water table or capillary water of rise in the subgrade to within 1 foot of the bottom of the base course. It is recommended that the finished grade of military roads and runways be at least 5 feet above the ground-water table. The condition of existing open channels should be investigated to find out if the ground-water table can be lowered by cleaning. Roots, vegetation, or other obstacles may have impeded the flow of the stream to such an extent that the adjacent areas have become saturated. Clearing out such obstructions may lower the flow line of the affected channels. Where no usable existing channels are found, new channels can be constructed. The depth of these channels is determined by the depth to which the ground-water table is to be lowered. For the drainage system to be most effective, the subgrade should be composed of relatively pervious materials which promote the passage of ground water from the subgrade into the ditch. It may be necessary to augment this method of drainage by the use of underground perforated pipe or open-joint filter drains.

(5) *Intercepting drainage.* Intercepting subsurface drainage is required where ground-water flow or seepage raises the ground-water table to within 1 foot of the base course, or within 5 feet of the finished grade of road or runway surface. Intercepting drainage is also provided to collect water from springs in subgrade ex-

cavation and to intercept ground water flowing on impervious strata. This may occur in cuts. These drains may consist of either subsurface drain pipe or open channels. Figure 5 shows a typical pipe installation for an airfield. Note that the drains are usually located along the line of the side channels and should be placed at least 5 feet below the surface. The bottom of the side channel is made impervious to avoid a combination drain, which, as indicated in (6) below, is not recommended.

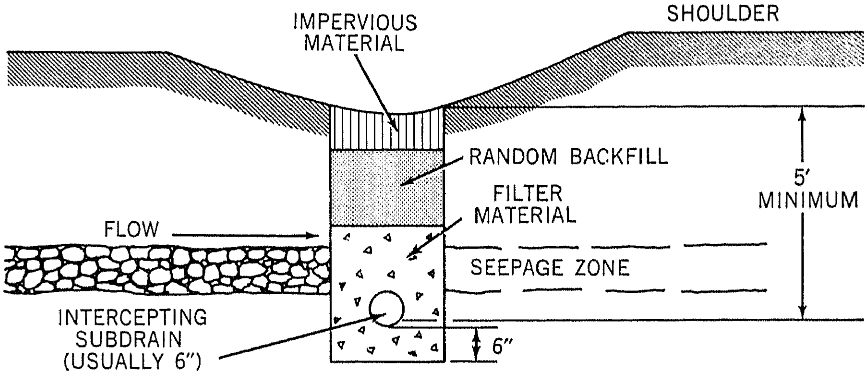


Figure 5. Typical installation of an intercepting drain (airfield).

(6) *Combination drains.* The use of combination drains which attempt to handle both surface runoff and subsurface water by use of the same pipe system is not recommended. Surface runoff very often carries sediment and soil from the drained area into the system, with the result that flow stoppages occur. In view of this, subsurface drainage systems using some form of piping are generally sealed so that no surface runoff may enter. The only drainage system which will handle both surface runoff and subsurface water satisfactorily is the open channel or ditch.

(7) *Blind or French drains.* Blind or French drains are constructed by filling a ditch or trench with broken or crushed rock. The top surface of the rock may be left exposed so that the trench will act as a combination drain or the rock may be covered by a relatively impervious soil so that no surface water can penetrate. The latter is the general practice. In general, French drains are not recommended for permanent construction because they

have a tendency to silt up with prolonged use. In theater-of-operations construction, however, such drains are often used as a substitute for perforated or open-joint pipe because of logistical limitations on piping or on filter materials suitable for use with such piping.

(8) *Vertical (dry) wells.* Vertical wells are sometimes constructed to permit trapped subsurface water to pass through an impervious soil or rock layer to a lower, freely draining layer of soil. If drainage is obstructed, additional wells are driven, or the pocket is drained with a lateral subdrain system which is easily maintained. Vertical wells are often used in northern latitudes where deep freezing is the rule, to permit fast runoff from melting snow to get through the frozen soil and reach a pervious stratum. The bottom of these wells is treated with calcium chloride or a layer of hay to prevent freezing.

c. Design and Construction of Subdrains. Subdrains are usually placed 2 feet or more below the surface. Subdrains are usually constructed with some type of pipe, protected by a layer of filter material (*d* below) at least 6 inches in depth. Types of pipe used for subdrain include the following:

(1) Perforated pipe is often used. In cases where the perforations do not extend completely around the circumference of the pipe, the pipe is generally laid with the holes down and with the joints closed. Materials used in the manufacture of this type of pipe are corrugated metal, cast iron, vitrified clay, bituminized fiber, asbestos cement, and nonreinforced concrete.

(2) Bell and spigot pipes can be laid with open joints. Collars are not needed over the joints if the filter material has been properly designed. This type of pipe is generally made of vitrified clay, reinforced or nonreinforced concrete, or cast iron.

(3) Cradle-invert pipe is designed so that water enters the pipe at bell and spigot joints which provide a gap around the bottom semicircumference and a slot across the flat top surface of the pipe. Cradle-invert pipe is generally made of vitrified clay or cast iron.

(4) Porous-concrete pipe is laid with closed joints and collects water by seepage through the wall of the pipe. It should not be used where sulfated waters may cause disintegration of the concrete.

(5) Drain tile is laid with butt joints slightly separated to permit collection of water through the joint. Because of

its low resistance to high impact loads, drain tile is not recommended for use on airfields. Materials commonly used in manufacture of the tile are clay or concrete.

d. Filter Material for Subdrains. The material which is used to backfill around the pipe in a subsurface drain, and for similar purposes, is generally known as a filter. It must be very carefully selected if the drain is to fulfill its intended function, particularly where the drain is located in soils with appreciable quantities of cohesionless or slightly plastic fines. The reason for this is that, as the water moves toward the drain, seepage pressures are produced which tend to move the soil grains toward the pipe. The finer cohesionless materials may be washed out, which may lead to the settlement of the soil surrounding the pipe, the formation of erosion channels in the drained soil, or clogging of the drain by the infiltration of small soil particles. Also the filter material must be permeable enough to allow free ground water to reach the pipe at a satisfactory rate. In order to prevent failures of the types just listed, various criteria have been developed. These criteria are based upon the mechanical-analysis soil curves, and have proved effective in practice. These criteria include:

(1) To prevent the movement of particles from the protected soil into or through the filter or filters, the following conditions must be satisfied:

$\frac{\text{15-percent size of filter material}}{\text{85-percent size of protected soil}}$ is less than or equal to 5, or $\frac{D_{15}}{D_{85}} \leqq 5$ and

$\frac{\text{50-percent size of filter material}}{\text{50-percent size of protected soil}}, \frac{D_{50}}{D_{50}}$ is less than or equal to 25.

Note. By D_{15} is meant the grain diameter which is larger than 15 percent of the soil grains; that is, the 15-percent passing size obtained from the grain-size distribution curve. Similarly for D_{50} and D_{85}. For most fine-grained soils, standard concrete sand makes a satisfactory filter material. When such sand contains a sufficient amount of fine gravel sizes, and when it is used for pipes with small openings, it is safe against infiltration into the pipe.

(2) To permit free water to reach the pipe, the filter material must be many times more pervious than the protected soil. This condition is fulfilled when the following requirement is met.

$\frac{\text{15-percent size of filter material}}{\text{15-percent size of protected soil}}$ is greater than or equal to 5.

(3) To prevent clogging the pipe with filter material moving through the perforations or openings, the following limiting requirements must be satisfied:

For slotted openings:

$$\frac{\text{85-percent size of filter material}}{\text{slot width}} \text{ is greater than 1.2.}$$

For circular holes:

$$\frac{\text{85-percent size of filter material}}{\text{hole diameter}} \text{ is greater than 1.0.}$$

(4) The coefficient of uniformity (C_u) of the filter material should be less than 20 to prevent segregation of the material during placement. The coefficient of uniformity is defined by the following relationship:

$$C_u = \frac{\text{60-percent size of filter material}}{\text{10-percent size of filter material}}$$

(5) The above criteria will be used when protecting all soils except medium to highly plastic clays without sand or silt partings, which by the above criteria may require multiple-stage filters. For these clay soils the D_{15} size of the filter may be as great as 0.4 mm and the D_{50} criteria given in (1) above will be disregarded. The relaxation in criteria for protecting medium to highly plastic soils will allow the use of a one-stage filter material; however, the filter must be well graded as well as meet the requirements of (4) above.

(6) It is suggested that the following criteria be applied in designing filters for porous concrete pipe:

$$\frac{\text{15-percent size of aggregate in porous pipe}}{\text{85-percent size of filter adjacent to porous pipe}} \leqq 5$$

(7) In the event that laboratory tests are unavailable a good grade of concrete sand may be used as a filter.

e. Theater-of-Operations Considerations. It should be noted that subsurface drainage other than that provided by open channels and ditches is seldom employed in theater-of-operations construction.

13. Erosion Control

a. Description. Erosion control is required not only to avoid the creation of traffic hazards, but also to facilitate keeping drainage structures effective and clear with a minimum of maintenance. Erosion will occur at any point where the force of moving water exceeds the cohesive strength of the material with which the water is in contact. Basically, the numerous methods of control depend

on dissipating the energy of the water or on providing an erosion-resistant protective surface.

b. Checkdams.

(1) *Use.* On sidehill cuts and steep grades, checkdams are placed in side ditches to slow the water and prevent it from washing out the road. Checkdams are not used when the original ditchline grade exceeds 5 percent because this would require placing the dams too close together. In such cases, side ditches should be paved with boulders or timbers. Also, if erosion is a major problem, as in sand cuts, paved ditches are better than checkdams (*c* below).

(2) *Construction.* Checkdams may be constructed of timber, sandbags, concrete, rock, or similar materials. They must extend at least 24 inches into the sides and bottom of the ditch. The bottom edge must extend at least 24 inches into the ground. Height of top of ditch above top of checkdam should be at least 12 inches. Effective height of the checkdam should be at least 12 inches but not more than 36 inches. Side slopes of the ditch above and immediately below checkdams will require protection from erosion. An apron extending at least 4 feet from the face of the checkdam must be provided to prevent scouring. A weir notch or discharge slot at the top of the checkdam must be provided, with a capacity big enough to discharge anticipated runoff, or water will back up and start cutting around the edges of the checkdam. The slot is always constructed 6 inches deeper than the depth of flow as a safety factor (fig. 6). Typical checkdam construction is illustrated in figure 7.

(3) *Spacing.* Checkdams are spaced close enough to produce about a 40 or 50 to 1 slope. The drop for the checkdam

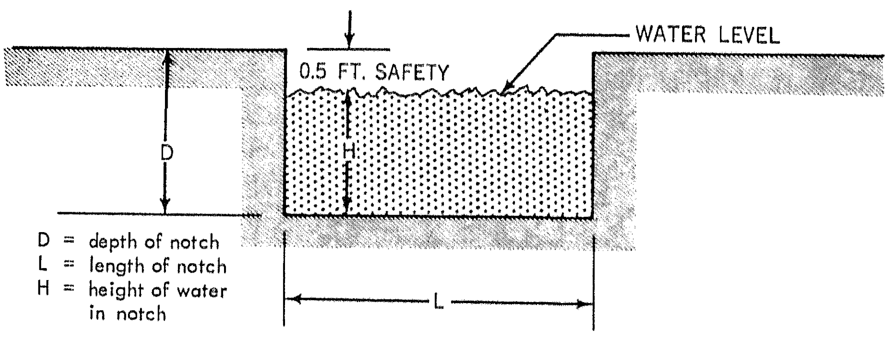


Figure 6. Weir notch with safety factor.

should be at least 1 foot and not more than 3 feet. Aprons to prevent scour on the downstream side of the checkdam should extend approximately 3 feet for each foot of vertical drop between the bottom of the weir notch and the top of the apron. The formula for computing the space or distance between dams is as follows:

$$S = \frac{100 \times H}{A - B}, \text{ in which}$$

S = distance between checkdams in feet
H = height of drop in feet for each checkdam
A = original slope of water in ditch in percent
B = final slope of water in ditch in percent

To locate the dams on the ground divide the length of the ditch by the spacing. The first dam is always at the bottom of the adverse grade. For example, suppose it is desired to reduce the slope of an existing 550-foot ditch from 3 percent to 1 percent with checkdams. The checkdams are to have 1 foot of drop per dam. See figure 8 and table III.

$$S = \frac{100 \times 1}{3 - 1} = \frac{100}{2} = 50 \text{ ft}$$

Number of dams required =

$$\frac{550}{50} = 11$$

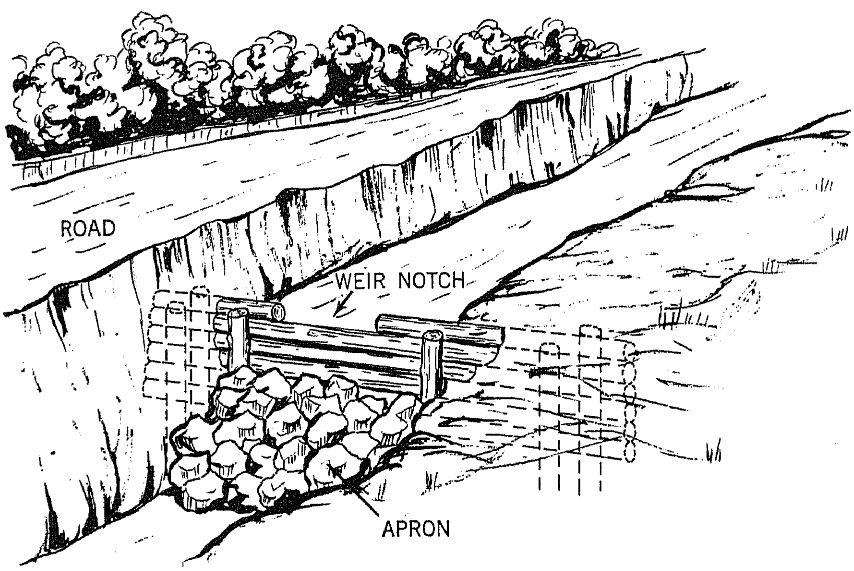


Figure 7. Typical checkdam construction.

The 11 dams will take up 11 feet of the difference in elevation of the ditch. $550 \times .03 = 16.5$ feet total drop in the ditch.

$$16.5 - 11 = 5.5 \text{ feet}$$

$$\frac{5.5}{550} \times 100 = 1 \text{ percent final slope of ditch}$$

Table III. Spacing of Checkdams, in Feet

Difference in slope ($A-B$) in percent	Values of H, in feet				
	1	1½	2	2½	3
½	200	300	400	500	600
1	100	150	200	250	300
1½	67	100	133	167	200
2	50	75	100	125	150
2½	40	60	80	100	120
3	33	50	67	83	100
3½	29	43	57	71	86
4	25	38	50	63	75
4½	--------	33	44	56	67

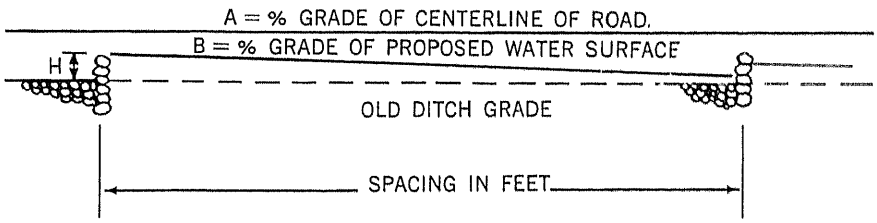


Figure 8. Basis for computing checkdam spacing.

c. Paving. Paving with asphalt or concrete is a method of providing an erosion-resistant surface in gutters, ditches, and pipe outfalls, and is required in ditches on grades in excess of 5 percent. Paving is not generally recommended because it is expensive and the ditch water still retains its erosive energy which must be controlled at some point farther along the system.

d. Sod Strips. For temporary roads, a strong sod strip is much easier to install and should prove adequate. The sod strip, not less than 12 inches wide and 2 or 3 inches thick, is firmly tamped in place on the bottom and sides of the ditch on the upstream side of the checkboard at the usual intervals for checkdams. Watering should be provided, if necessary to insure establishment of the sod. The effectiveness of sodding is greatly increased by the addition of the checkboard as shown in figures 9 and 10. In this case, the following design features should be considered:

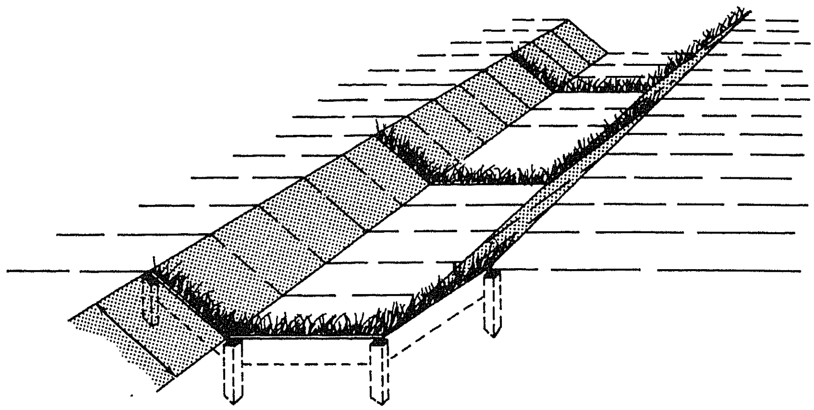


Figure 9. Perspective of drainage ditch with erosion checks.

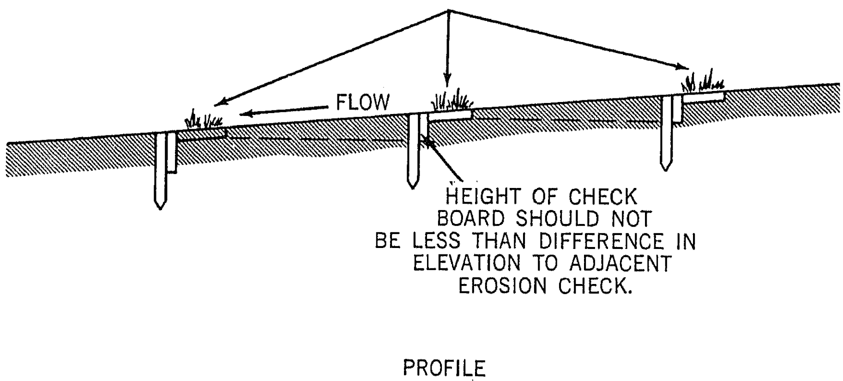


Figure 10. Profile of erosion checks.

(1) Recommended difference in elevation between checks is 6 inches and is never more than 12 inches.

(2) To prevent undermining, depth of checkboard should be more than the difference in elevation between checks.

(3) Particular care should be taken to make the checkboard level, to prevent uneven distribution of flow over the sod.

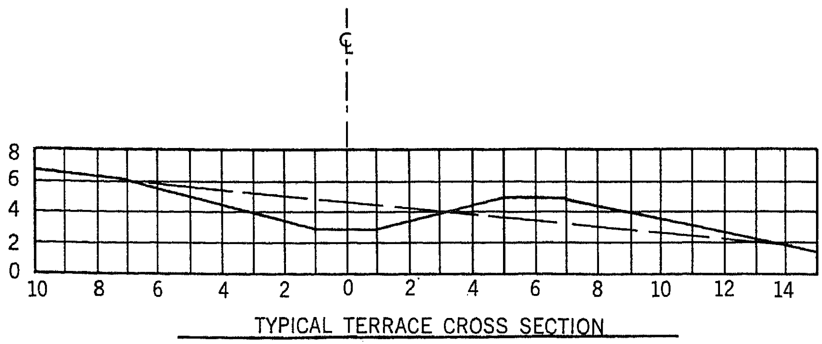
(4) The side checkboards should be extended up the channel sides enough to prevent end flow.

e. Expedient Materials. Turfing, placing riprap, and spreading rubble are control methods designed to cause turbulence in order to dissipate the energy of flow in channels, ditches, and pipe outfalls.

f. Terracing. Erosion control in nonuse areas may frequently be accomplished by terracing in conjunction with a well-developed turfing program. The terrace consists of a low, broad-based earth levee constructed approximately parallel to the contours. It is designed to intercept overland flow before it can cause erosion, and to conduct it to a suitable discharge point. A cross section through a typical terrace is shown in figure 11. The slopes should be sufficiently flat to facilitate operation of mowing equipment or any other traffic use to which the area may be subjected. The vertical spacing of terraces is a variable, dependent on the erosive character of the soil and on prevailing land slope. A guide to average vertical spacing is indicated in figure 11. The gradient of the channel along the upper side of the terrace must be selected to prevent erosion and yet provide positive drainage. Suggested values for channel gradients are also indicated in figure 11. Terrace lengths greater than 1,500 feet are not advocated. Special attention should be given to establishing a vigorous turf on terraces and channels because of potential danger arising from unprotected excavations, particularly in silty soils.

14. Culvert Considerations

a. Characteristics of Culverts. With the exception of open-top log, open-top timber, and open-top rock culverts discussed in paragraph 18 and illustrated in figures 2 and 28, culverts are closed waterways placed through embarkments or fills. Culverts are used to provide cross drainage at low points in a fill section, to provide ditch relief, and to continue side ditches at intersections. Their efficiency, low cost, strength, and ease of construction make them more desirable than small bridges for a wide range of construction. Generally, culverts should be used where bedding conditions are favorable, cover is adequate, and jamming by debris and/or ice is not likely, and where the quantity of flow will not create excessive velocity and erosion. Culverts are designed to carry the maximum amount of water that is likely to flow in the drainage channel. Normally, they are constructed on grades fixed between the minimum required for the free flow of water and the maximum determined by the design grades of the road surface and the elevation of the outfall. To prevent the accumulation of sediment in pipes, the gradient should be sufficient to give the water a velocity of at least 2.5 feet per second (par. 21). Pipe culverts, in the form of nestable corrugated steel pipe, are standard items of engineer supply. Diameters commonly range from 8 to 36 inches, but sizes as large as 84 inches may be supplied to theaters of operations. In addition to the nestable corrugated culvert, box culverts of wood or concrete can be constructed. Im-



VERTICAL SPACING OF TERRACES

AVERAGE LAND SLOPE (PERCENT)	VERTICAL DROP (FEET)
2	2.50
4	3.00
6	3.50
8	4.00
10	4.50
12	5.00
14	5.50

GRADIENTS FOR VARIABLE GRADE TERRACE

LENGTH OF TERRACE (FEET)	TERRACE CHANNEL GRADE (PERCENT)
0 - 300	0.10
300 - 600	0.15
600 - 900	0.20
900 - 1200	0.30
1200 - 1500	0.40

Figure 11. Details for a typical terrace.

provisation, using locally available materials, such as oil or asphalt drums, is frequently desirable and often saves time and transportation. In some areas, large water pipes, sewer pipes, and other civilian materials may be used. A full discussion of culverts is given in paragraphs 15 through 21.

b. Length. The length of a culvert is determined by the width of the embankment at the location where the culvert is to be installed. Culverts must be long enough to prevent earth from being worked into them from the fill and also to prevent the roadbed or embankment from being scoured by the water as it leaves the culvert (fig. 12). Usually, culverts should be long enough to extend completely through fills to the point where the fill slope meets the

ground or stream bed level. In some instances, such as where grades are steep, excessive lengths can be avoided by placing the outlet above the toe of fill or stream level. For a cut section, the normal length would be equal to the distance from the bottom of the ditch on the upstream side to the bottom of the ditch on the outlet side. To minimize scour at the downstream end, culverts should be 1 or 2 feet longer than required, with the added length on the discharge end. In some instances, it will be necessary to prevent scouring by construction of a toe ditch to carry the water off the slope. For lengths up to 30 feet, the use of pipes smaller than 12 inches in diameter should be avoided; for lengths over 30 feet, pipes smaller than 15 inches should be avoided. Small pipes clog easily and are difficult to maintain. If large headwalls are used, the length of the culvert may be shortened, but it usually takes less time, labor, and materials to build longer culverts.

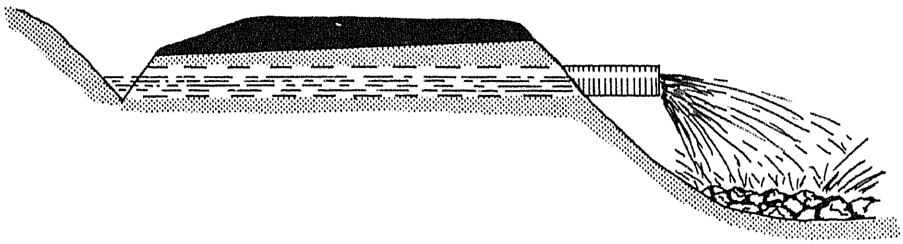


Figure 12. Culvert extended beyond fill to prevent erosion.

c. Strength. The culvert must be strong enough to carry the weight of the fill above it plus the weight of live loads that pass over the road. To calculate the strength of a culvert requires a knowledge of structural design which is beyond the scope of this manual. If well-nailed, spiked, driftpinned, bolted, or wired together, the culverts shown in figures 29 through 36 are structurally sound enough to carry combat traffic plus sand fills from 2 to 10 feet deep. Culverts are constructed to act as a single complete unit under loads. Tables IV, V, and VI give strength requirements of rigid and flexible pipes under various loads.

d. Alinement. Culverts are placed in natural drainage channels (fig. 13) unless such installations would require an unusually long culvert or produce a sharp bend in the channel on the upstream side. Alinement is not changed where the angle between the stream and the embankment is 45° or more. If a meandering stream is encountered, the culvert should be installed at the best possible location and the stream channel straightened as necessary

Table IV. Minimum Pipe Cover Requirements for Airfields

Type of Pipe	15,000-lb Single-Wheel Cover, ft, for Pipe Diam, in., of										25,000-lb Single-Wheel Cover, ft, for Pipe Diam, in., of									
	6	12	24	36	48	60	72	84	96	108	6	12	24	36	48	60	72	84	96	108
Standard strength clay pipe (ASTM C13–57T, AASHO M65–57, and Fed Spec SS–P–36 lb and Am 1).	2.0	2.5	3.5	2.5							2.5	3.5	4.3	3.0						
Extra strength clay pipe (ASTM C200–57T, AASHO M65–57, and Fed Spec SS–P–36 lb and Am 1).	1.5	2.0	2.0	2.0							1.5	2.5	2.0	2.5						
Standard strength nonreinforced-concrete pipe (ASTM C14–58, and Fed Spec SS–P–371a).	2.0	2.5	3.5								2.5	3.5	4.5							
Extra strength nonreinforced-concrete pipe (ASTM C14–58 and Fed Spec SS–P–371a).	1.5	2.0	2.0								1.5	2.5	2.5							
Reinforced-concrete culvert, storm drain, and sewer pipe (ASTM C76—59T and AASHO M170–57).																				
Class I						2.5	2.5	2.5	2.5	2.5						3.5	3.5	3.0	3.0	3.0
Class II		2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0		3.5	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5
Class III		2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0		3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0
Class IV		1.5	1.5	1.5	1.5	1.5	1.5	1.5				2.0	2.0	2.0	2.0	2.0	2.0	2.0		
Class V		1.5	1.0	1.0	1.0	1.0	1.0					2.0	1.5	1.5	1.5	1.5	1.5			
Corrugated metal pipe (Fed Spec QQ–C–806a and Am 1, AASHO M36–57, or AREA Spec 1–4–6, 1953).																				
18-gage	1.0	1.5									1.0	2.0								
16-gage	1.0	1.0	1.5								1.0	1.5	2.0							
14-gage		1.0	1.0	2.0								1.0	1.5	2.5						
12-gage			1.0	1.0	2.0								1.5	2.0	2.0					

Type of Pipe	100,000-lb Twin-Wheel Assembly Cover, ft, for Pipe Diam, in., of									
	6	12	24	36	48	60	72	84	96	108
10-gage			1.0	1.0	1.0	1.5	2.0			
8-gage				1.0	1.0	1.0	1.5	2.0		
Standard strength clay pipe (ASTM C13–57T, AASHO M65–57 and Fed Spec SS–P–36 lb and Am 1).	8.0	—	—	—						
Extra strength clay pipe (ASTM C200–57T, AASHO M65–57, and Fed Spec SS–P–36 lb and Am 1).	3.5	6.0	6.5	7.0						
Standard strength nonreinforced-concrete pipe (ASTM C14–58 and Fed Spec SS–P–271a).	8.0	—	—							
Extra strength nonreinforced-concrete pipe (ASTM C14–58 and Fed Spec SS–P–371a).	3.5	6.0	7.5							
Reinforced-concrete culvert, storm drain, and sewer pipe (ASTM C76–59T and AASHO M170–57).										
Class I						—	—	—	—	—
Class II		—	—	—	—	—	—	—	—	—
Class III		—	7.5	7.0	6.0	6.0	5.5	5.5	5.5	5.0
Class IV		5.0	4.5	4.0	4.0	4.0	3.5	3.5	5.0	
Class V		4.0	3.5	3.0	3.0	3.0	3.0			

Type of Pipe	265,000-lb Twin-Twin-Wheel Assembly Cover, ft, for Pipe Diam, in., of									
	6	12	24	36	48	60	72	84	96	108
10-gage			1.0	1.5	2.0	2.5	3.0			
8-gage				1.0	1.5	2.0	2.5	3.0		
Standard strength clay pipe (ASTM C13–57T, AASHO M65–57 and Fed Spec SS–P–36 lb and Am 1).	—	—	—	—						
Extra strength clay pipe (ASTM C200–57T, AASHO M65–57, and Fed Spec SS–P–36 lb and Am 1).	6.0	—	—	—						
Standard strength nonreinforced-concrete pipe (ASTM C14–58 and Fed Spec SS–P–271a).	—	—	—							
Extra strength nonreinforced-concrete pipe (ASTM C14–58 and Fed Spec SS–P–371a).	6.0	—	—							
Reinforced-concrete culvert, storm drain, and sewer pipe (ASTM C76–59T and AASHO M170–57).										
Class I						—	—	—	—	—
Class II		—	—	—	—	—	—	—	—	—
Class III		—	—	—	—	—	—	—	—	—
Class IV		9.5	8.0	7.5	7.0	6.5	6.0	6.0		
Class V		6.5	6.0	6.0	5.5	5.0	5.0			

Table IV. Minimum Pipe Cover Requirements for Airfields—Continued

Type of Pipe	100,000-lb Twin-Wheel Assembly Cover, ft, for Pipe Diam, in., of										265,000-lb Twin-Twin-Wheel Assembly Cover, ft, for Pipe Diam, in., of									
	6	12	24	36	48	60	72	84	96	108	6	12	24	36	48	60	72	84	96	108
Corrugated metal pipe (Fed Spec QQ–C–806a and Am 1, AASHO M36–57, or AREA Spec 1–4–6, 1953).																				
18-gage	2.5	3.0									3.5	4.0								
16-gage	2.0	3.0	5.0								3.0	3.5	7.0							
14-gage		3.0	4.0	5.5								3.5	6.0	9.0						
12-gage			3.5	4.5	5.5								4.5	8.0	9.0					
10-gage			3.0	4.0	5.0	5.5	6.0						4.0	7.0	8.5	9.0	9.5			
8-gage				3.5	4.0	5.0	5.5	6.0						6.5	8.0	8.5	9.0	9.5		

Type of Pipe	15,000-lb Single-Wheel Cover, ft, for Pipe Diam, in., of											25,000-lb Single-Wheel Cover, ft, for Pipe Diam, in., of										
	6	8	10	12	14	16	18	20	24	30	36	6	8	10	12	14	16	18	20	24	30	36
Asbestos-cement pipe (Fed Spec SS–P–331b).																						
Class 1500	1.5	2.0	2.0	2.5	3.0	3.5						2.0	2.5	2.5	3.5	4.0	4.5					
Class 2400	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5	3.5			1.5	2.0	2.0	2.5	2.5	2.5	3.0	3.5	4.5		
Class 3300	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.0	2.5		1.0	1.5	1.5	2.0	2.0	2.5	2.5	2.5	3.0	3.5	
Class 4000			1.0	1.0	1.5	1.5	1.5	2.0	2.0	2.0	2.5			1.5	1.5	2.0	2.0	2.0	2.5	2.5	3.0	3.0
Class 5000			1.0	1.0	1.0	1.5	1.5	1.5	1.5	2.0	2.0			1.0	1.5	1.5	2.0	2.0	2.0	2.0	2.5	2.5

Type of Pipe	100,000-lb Twin-Wheel Assembly Cover, ft, for Pipe Diam, in., of											265,000-lb Twin-Twin-Wheel Assembly Cover, ft, for Pipe Diam, in., of										
	6	8	10	12	14	16	18	20	24	30	36	6	8	10	12	14	16	18	20	24	30	36
Asbestos-cement pipe (Fed Spec SS-P-331b).																						
Class 1500	5.0	6.5	—	—	—	—						8.0	—	—	—	—	—					
Class 2400	3.0	4.0	5.0	6.0	7.5	—	—	—	—			5.0	6.5	—	—	—	—	—	—	—		
Class 3300	2.5	3.0	3.5	4.5	5.0	6.0	6.5	7.5	—	—		3.5	5.0	6.0	8.0	—	—	—	—	—	—	
Class 4000			3.0	3.5	4.0	5.0	5.0	6.0	7.5	—	—			5.0	6.0	7.0	8.0	—	—	—	—	—
Class 5000			2.5	3.0	3.0	3.5	4.0	5.0	5.5	7.5	—			4.0	5.0	5.5	6.5	7.0	8.0	—	—	—

Notes.

1. Except where individual pipe installation designs are made, cover for pipe beneath runways, taxiways, aprons, or similar traffic areas shall be provided in accordance with this table except as provided in note 7 for pipe underlying rigid pavements.

2. Cover for pipe in airfield nontraffic areas shall be designed for 15,000-lb single-wheel load.

3. Cover depths are measured from top of flexible pavement or unsurfaced areas to top of pipe.

4. Table to be used for both trench- and embankment-type installations.

5. Pipe produced by certain manufacturers exceeds the strength requirements established by the indicated standards. When additional strength is proved, the minimum cover may be reduced accordingly.

6. Design loads for reinforced-concrete box culverts may be determined from procedure suggested in PCA Bulletin No. 10, "Modern developments in reinforced concrete."

7. Pipe placed under airfield rigid pavements shall have a minimum cover, measured from the bottom of the slab, as follows:

Pipe Sizes, in.	Minimum Cover, ft, for Loads of 15-kip SW	25-kip SW	100-kip Twin assembly	265-kip Twin-Twin Assembly
6–60	0.5	0.5	1.0	1.0
66–108	1.0	1.0	1.5	1.5

8. Dashes (—) in table indicate allowable load is less than load on pipe. Blanks indicate that pipe is not specified by the applicable standards.

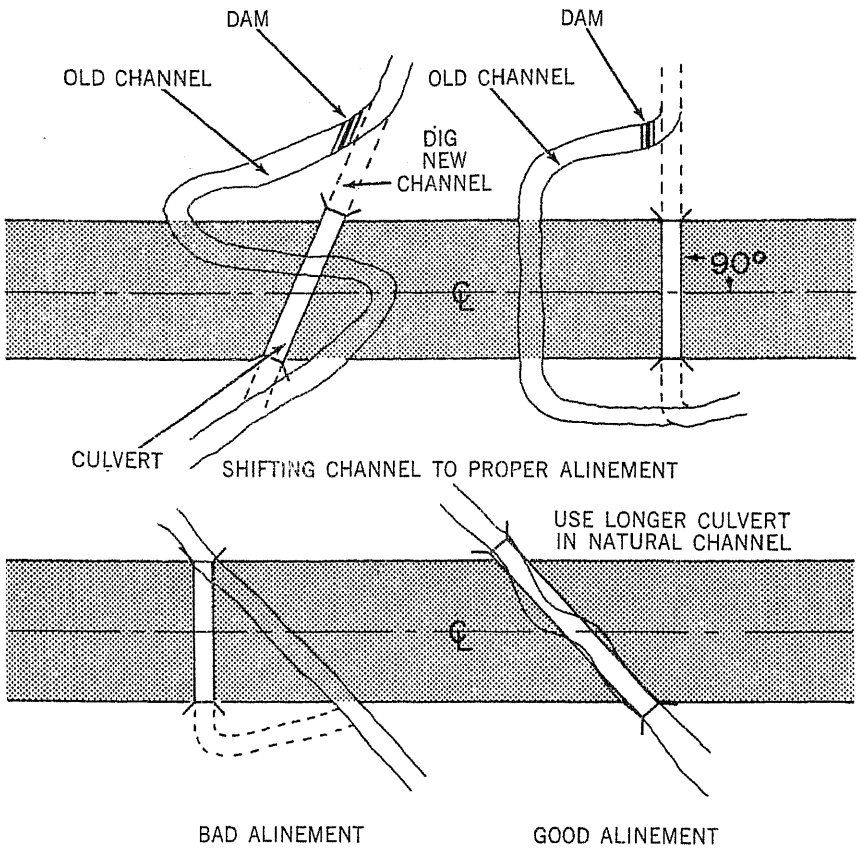


Figure 13. Alinement of culverts.

(fig. 13). Where old drainage channels are not encountered, culverts should be installed at right angles to the centerline of the traveled way, except that in sidehill cuts on steep grades, ditch-relief culverts should be installed at an angle of 60° to the centerline, to allow a more direct entrance of water into the culvert (fig. 15). For best hydraulic capacity culverts should not project at upstream ends.

e. Elevation. The bottom of the culvert at the inlet is placed on or below, but not above, the streambed. Filling under a culvert to bring it to grade should be avoided. If necessary, the inlet of the culvert is placed below the natural streambed and drop inlets or headwalls (par. 20) are used. Use of drop inlets requires periodic removal of accumulated sediment. At the outlet end, the bottom of the culvert should normally be at the elevation of the surface of the stream since it may fill with sediment if placed below the

surface. On sidehill cuts, it may be necessary to place the lower end of the culvert above the stream. In this case, spillways are constructed to prevent erosion and backwash, or the culvert is extended beyond the fill and above the ground (fig. 12).

f. Slope. Normally culverts are constructed at the grade of the natural and artificial drainage channels which discharge into them. It is generally desirable to use grades of from 2 to 4 percent. However, in extreme cases, where the fall of the land will permit, 0.5 percent may be used as an absolute minimum. Velocities (par. 21) should not be greater than 8 feet per second to avoid scouring, nor less than 2.5 feet per second to avoid sedimentation. Changes in the grade of the culvert should be avoided, but when changes in grade are unavoidable, the culvert should be designed so that the steepest grade is on the outlet end. When the outlet is below the ground surface in flat terrain, water should be removed in a ditch section adequate to prevent overflow until natural ground elevation is reached. In digging culvert ditches or installing culverts to grade, a reference string can be used. On a continuation of the culvert centerline, long stakes are driven about 1 foot outside the inlet and outlet ends of the culvert. These stakes are marked at a given distance above the inlet and outlet ends of the culvert and a string is stretched between these marks (fig. 14). Slope of the culvert may also be established by the use of batter boards, as described in TM 5–233.

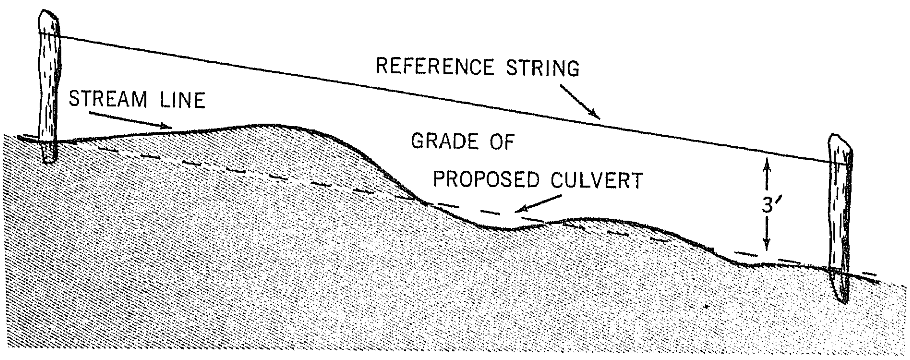


Figure 14. Establishing grade of culvert with reference string.

g. Spacing. Culverts should be located wherever natural drainage channels are large enough to require cross drainage. On sidehill roads or wherever roads intercept surface water either in cut or in fill, the water is drained to the low side of the road and, if possible, away from the road by ditch-relief culverts (fig. 15). On 8-percent grades, ditch-relief culverts should be placed about 300

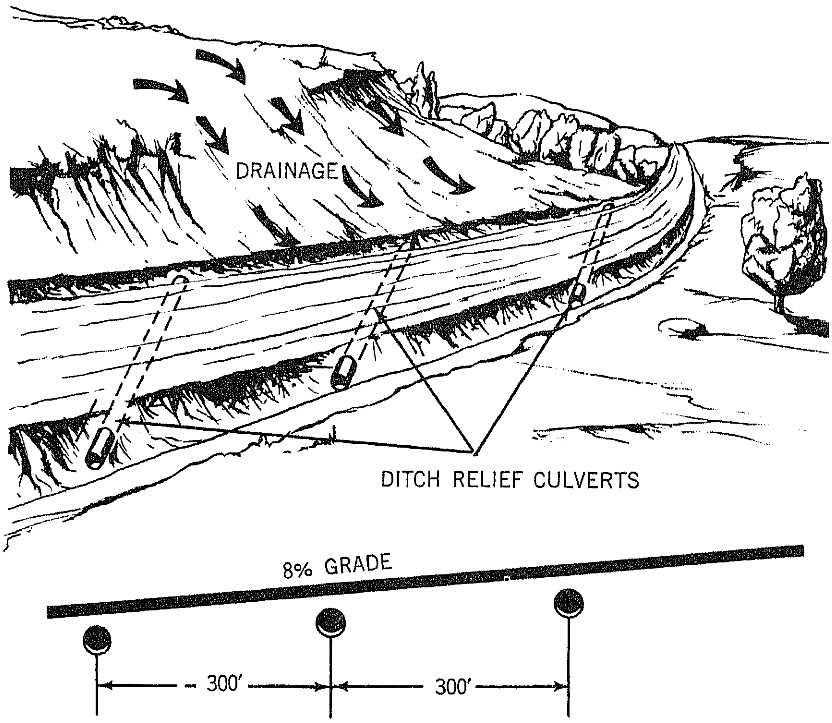


Figure 15. Spacing of ditch-relief culverts.

feet apart; on 5-percent grades, 500 feet apart. The distance between pipes in multiple-pipe culverts is at least one-half the diameter of the pipe (fig. 16).

h. Foundation. Culverts are constructed on a firm, well-compacted soil foundation, except that box or arch culverts may be placed on rock foundation when suitable rock is encountered. The foundation is always shaped to fit, or bed, at least one-tenth of the outside diameter of the pipe (fig. 17). In addition, foundations for pipe culverts are generally curved upward along the culvert centerline to correct for expected settlement and to insure tightness in the lower half of the joints. Sometimes cradles are built to provide proper support and to avoid uneven settlement (fig. 18). If the bearing strength of the soil is completely inadequate, footings are placed to distribute the load. The spread footing for timber culverts, shown in figure 19, can be adapted for all types of culverts and various types of soil.

i. Backfill. Dirt is backfilled and tamped by hand or mechanical tamper to one-half the culvert depth, or to a depth sufficient to

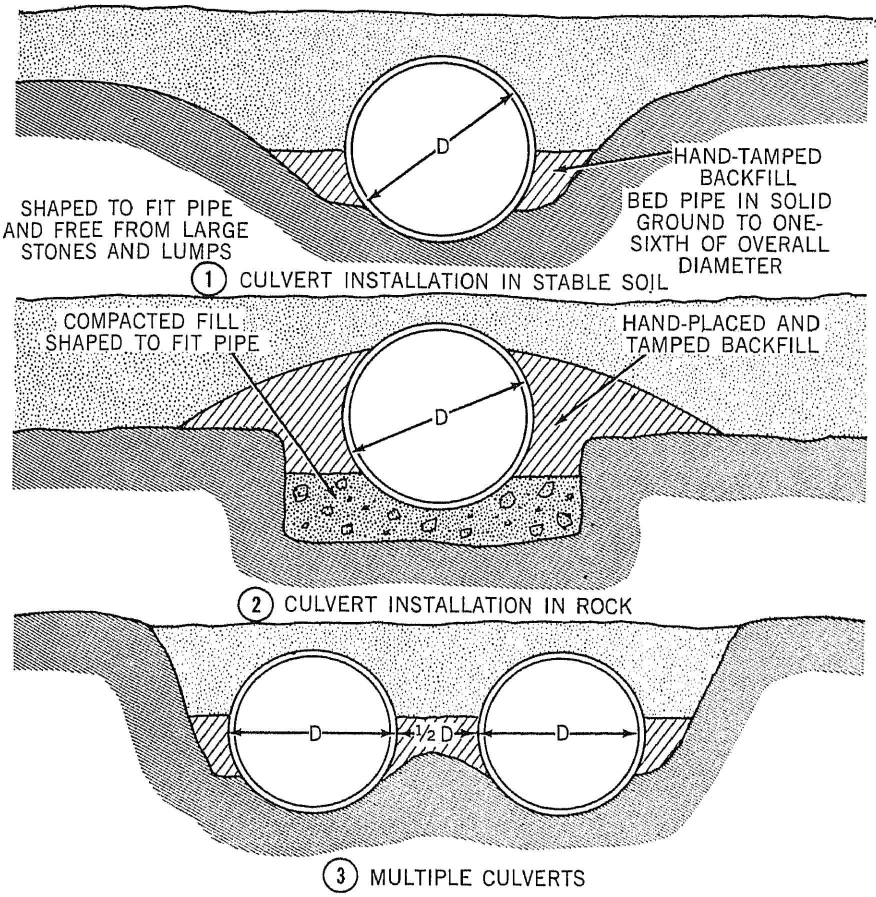


Figure 16. Culvert installation.

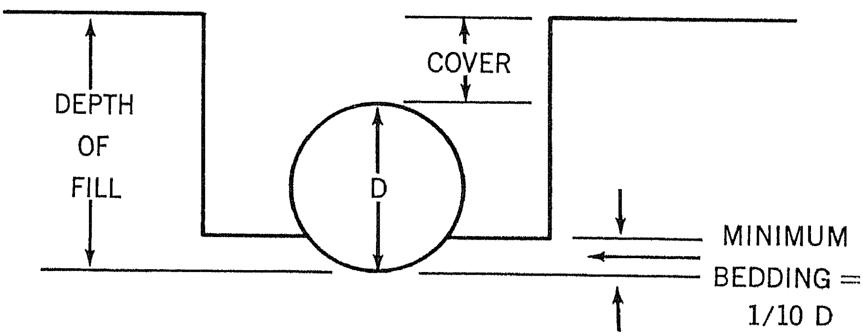


Figure 17. Culvert bedding.

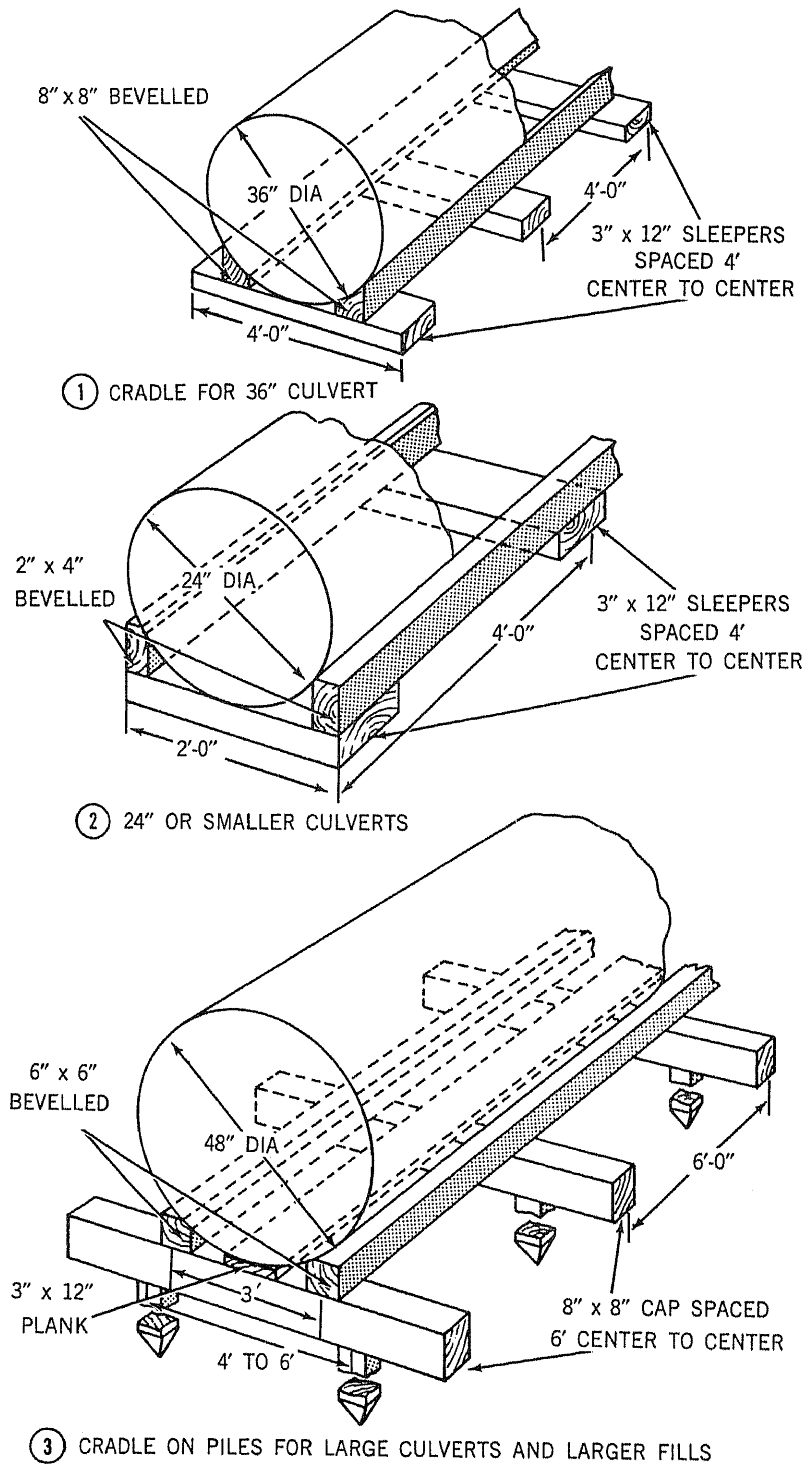


Figure 18. Cradles for pipe culverts.

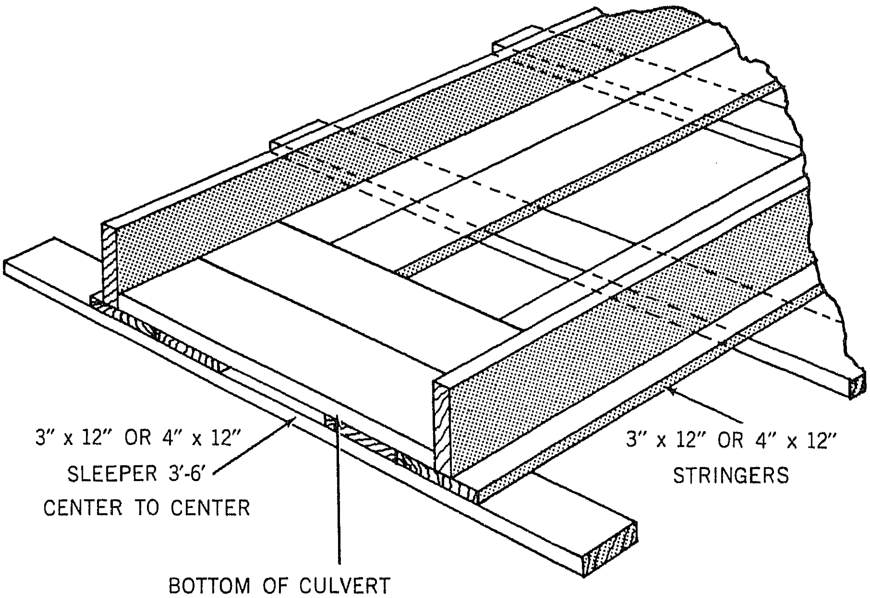


Figure 19. Spread footing for timber culverts.

hold the culvert in place. The backfill is then completed with bulldozers and other equipment, but tamping is by hand or mechanical tamper to at least 12 inches above the culvert.

j. Cover. Cover is the depth of compacted earth from the top of the culvert to final grade of overlying material. Depth of fill is measured from the culvert invert to the edge of the shoulder as shown in figure 17.

(1) *Minimum.* A general rule for the minimum cover for road culverts is one-half the pipe diameter, or 12 inches, whichever is greater. However, where greater or lesser minimums are shown in table V the tabular values will govern. The minimum cover for pipe culverts for airfields is selected from table IV, which is based on the weight of the aircraft. During construction, where heavy construction equipment will be crossing frequently, adequate covers must be provided to protect culverts from damage.

(2) *Maximum.* Maximum cover for rigid pipe road culverts is given in table V. The maximum recommended height of fill for corrugated metal pipe is given in table VI.

k. Critical Slope. For discussion of critical slope in culverts see paragraph 21.

Table V. Minimum and Maximum Cover for Rigid Pipe Culverts, Roads

Specification	Kind of pipe	Class of pipe	12″			15″			18″			24″			36″			48″			60″			72″		
			Strength test load 3-edge lbs. per lin ft. of pipe	Min. cover in feet	Max. cover in feet	Strength test load 3-edge lbs. per lin ft. of pipe	Min. cover in feet	Max. cover in feet	Strength test load 3-edge lbs. per lin ft. of pipe	Min. cover in feet	Max. cover in feet	Strength test load 3-edge lbs. per lin ft. of pipe	Min. cover in feet	Max. cover in feet	Strength test load 3-edge lbs. per lin ft. of pipe	Min. cover in feet	Max. cover in feet	Strength test load 3-edge lbs. per lin ft of pipe	Min. cover in feet	Max. cover in feet	Strength test load 3-edge lbs. per lin ft. of pipe	Min. cover in feet	Max. cover in feet	Strength test load 3-edge lbs. per lin ft. of pipe	Min. cover in feet	Max. cover in feet
			1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
AASHO M41–42	Rein. concrete	St. St.	2250	2′	10′	2625	2′	10′	3000	2′	10′	3000	2½′	7′	4050	2′	6′	5400	2′	7′	6000	1½′	7′	6600	1′	6′
AASHO M41–42	Rein. concrete	Ex. St.	--	--	--	--	--	--	--	--	--	4000	2′	10′	6000	1½′	10′	8000	1′	11′	9000	1′	10′	9900	1′	10′
AASHO M65–42	Clay	Ex. St.	2250	2′	10′	2750	2′	10′	3300	2′	11′	4400	1½′	11′	6000	1½′	10′									
ASTM C76–41	Rein. concrete	St. St.	2250	2′	10′	2625	2′	10′	3000	2′	10′	3000	2½′	7′	4050	2′	6′	5400	2′	7′	6000	1½′	7′	6600	1′	6′
ASTM C76–41	Rein. concrete	Ex. St.	--	--	--	--	--	--	--	--	--	4000	2′	10′	6000	1½′	10′	8000	1′	11′	9000	1′	10′	9900	1′	10′
Fed. Spec. SS–P–136a	Clay	Ex. St.	2250	2′	10′	2750	2′	10′	3300	2′	11′	4400	1½′	11′	6000	1½′	10′									

Table V. Minimum and Maximum Cover for Rigid Pipe Culverts, Roads—Continued

Pipe Diam in.	Asbestos-cement pipe (Fed Spec SS-P-331b)				
	Trench condition			Positive projection condition	
	Min cover ft DL+LL	Max cover ft DL+LL	Max cover ft DL Only	Min cover ft DL+LL	Max cover ft DL+LL
	Class 1500				
6	1.5	NL	NL	1.5	21.0
8	2.0	NL	NL	2.0	17.0
10	2.0	17.0	17.0	2.0	13.0
12	2.5	11.0	11.5	2.5	11.0
14	3.0	9.0	9.5	3.0	9.0
16	3.5	7.0	8.5	3.5	7.0
	Class 2400				
6	1.0	NL	NL	1.0	34.0
8	1.5	NL	NL	1.5	27.0
10	1.5	NL	NL	1.5	21.0
12	2.0	NL	NL	2.0	18.0
14	2.0	30.0	30.0	2.0	15.0
16	2.0	19.0	19.0	2.0	14.0
18	2.5	13.0	13.0	2.5	13.0
20	2.5	11.0	11.5	2.5	11.0
24	3.5	8.0	9.0	3.5	8.0
	Class 3300				
6	1.0	NL	NL	1.0	45.0
8	1.0	NL	NL	1.0	35.0
10	1.0	NL	NL	1.0	29.0
12	1.5	NL	NL	1.5	24.0
14	1.5	NL	NL	1.5	21.0
16	2.0	NL	NL	2.0	18.0
18	2.0	34.0	34.0	2.0	17.0
20	2.0	22.0	22.0	2.0	15.0
24	2.0	15.0	15.0	2.0	13.0
30	2.5	10.0	10.5	2.5	10.0
	Class 4000				
10	1.0	NL	NL	1.0	35.0
12	1.0	NL	NL	1.0	29.0
14	1.5	NL	NL	1.5	25.0
16	1.5	NL	NL	1.5	22.0
18	1.5	NL	NL	1.5	20.0
20	2.0	NL	NL	2.0	18.0
24	2.0	22.0	22.0	2.0	15.0
30	2.0	13.0	13.0	2.0	13.0
36	2.5	10.0	10.5	2.5	10.0

Table V. Minimum and Maximum Cover for Rigid Pipe Culverts, Roads—Continued

Pipe Diam in.	Asbestos-cement pipe (Fed Spec SS-P-331b)				
	Trench condition			Positive projection condition	
	Min cover ft DL+LL	Max cover ft DL+LL	Max cover ft DL Only	Min cover ft DL+LL	Max cover ft DL+LL
			Class 5000		
10	1.0	NL	NL	1.0	42.0
12	1.0	NL	NL	1.0	35.0
14	1.0	NL	NL	1.0	31.0
16	1.5	NL	NL	1.5	27.0
18	1.5	NL	NL	1.5	24.0
20	1.5	NL	NL	1.5	22.0
24	1.5	42.0	42.0	1.5	19.0
30	2.0	20.0	20.0	2.0	15.0
36	2.0	14.0	14.0	2.0	13.0

Notes.

NL equals no limit.

Values based on the following assumptions:

1. Earth load only, load distribution due to rigid pavement not considered.
2. Average soil conditions at 120 lbs. per cubic foot.
3. Pipe laid in accordance with first class pipe laying conditions.
4. Test loads in pounds per lin. ft. of pipe are for 3-Edge Bearing Method, for clay and nonreinforced concrete pipe, and the load on reinforced concrete pipe to produce a crack on 0.01 inch.
5. Safe load computed by test load multiplied by 1.875 and divided by a factor of safety of 1.5.
6. Wheel load of 16,000 lbs., plus impact factor of 50 percent for cover 0.0′ to 2.0′, inc., and 25 percent for cover 2.5′ and more.
7. Computations for live load based on three feet length of pipe.
8. Assumed culvert pipe laid with 0.7 projection (p) of pipe above solid (natural soil) surface and with embankment fill with a settlement-deflection ratio r_{sd} = 0.7, equivalent to 0.5 projection condition.
9. For cover over pipe not exceeding 2 feet it is considered advisable to use pipe not less than 3 feet in length, and the minimum cover here shown shall not be reduced for lengths of individual pipe that are more than 3 feet.
10. For conditions other than as stated above the depth of safe minimum and maximum cover may vary depending upon conditions, and due allowances shall be made for such variations.
11. Minimum and maximum cover for intermediate and larger sizes than those shown in table shall be determined similarly.
12. Where cover over pipe is less than the minimum, or greater than the maximum shown, the pipe herein indicated shall be reinforced with concrete cradle or otherwise safeguarded, or stronger pipe specified.

Table VI. Permissible Maximum Cover for Corrugated Metal Pipe

Diameter in.	Permissible maximum cover, ft.									
	Circular section					Vertical elongated				
	Gage					Gage				
	16	14	12	10	8	16	14	12	10	8
8	80									
10	60	80								
12	60	70	80							
15	50	70	80							
18	40	60	80							
21	35	50	80							
24	15	45	70	80						
30	------	30	45	70	80					
36	------	15	30	45	70					
42	------	------	25	35	60					
48	------	------	20	25	35	------	------	25	70	80
54	------	------	15	20	30	------	------	20	50	80
60	------	------	------	15	25	------	------	------	45	80
66	------	------	------	15	20	------	------	------	35	70
72	------	------	------	10	15	------	------	------	25	60
78	------	------	------	------	------	------	------	------	------	25
84	------	------	------	------	------	------	------	------	------	20

Notes.

1. Except for gage tables, corrugated metal pipe shall conform to the requirements of Federal Specification QQ-C-806a and Aml, AASHO Standard M36-57, or to AREA Specification 1-4-6 (1953).
2. Table to be used for normal installation conditions, and for loads ranging from dead (earth) load only to dead load plus H-20-44 live load. H-20-44 live load is equivalent to a 32,000-lb axle load.
3. Vertical elongation shall be accomplished by either shop fabrication or field strutting, and shall generally be 5% of the pipe diameter.
4. Both circular and vertically elongated pipe placed under rigid pavements shall have a minimum cover, measured from the bottom of the slab, of 6 in. for pipe sizes up to 60 in. and 12 in. for sizes over 60 in.
5. Both circular and vertically elongated pipe placed under flexible pavements and in unsurfaced areas shall have a minimum cover, measured from the top of the surface, of 12 in. for pipe sizes up to 60 in. and 18 in. for sizes over 60 in.

15. Pipe Culverts

Pipe culverts consist of vitrified clay, asbestos-cement, reinforced or nonreinforced concrete, and either preformed (riveted) or nestable corrugated metal pipe. Choice of type will be limited to the types available, but reinforced concrete and corrugated metal pipes are preferred.

a. Vitrified Clay, Concrete, and Asbestos Cement. For clay, concrete, and asbestos-cement pipe, see paragraph 14*c* relative to strength requirements and paragraph 14*j* relative to cover. Joints for clay and concrete culverts shall be made with cement mortar, mixed 1 part cement and not more than 2 parts sand. Joints for asbestos-cement pipe shall be made with rubber sealing rings.

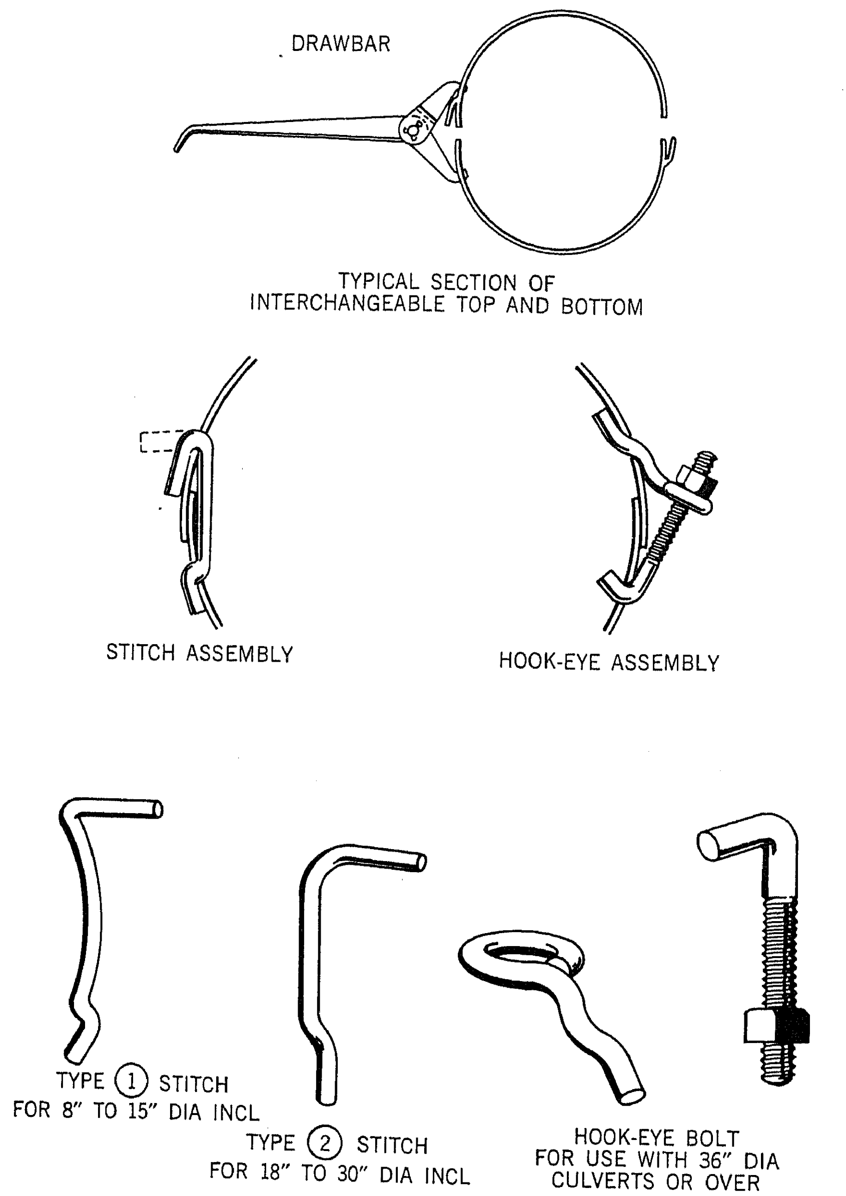


Figure 20. Some of the tools and fittings used in assembling the notched-type nestable pipe.

b. Corrugated Metal. Corrugated metal pipes are acceptable in all sizes, but preformed pipes are preferred to nestable field-assembled pipes. The latter are of two types: notched, having a notched edge and a plain edge; and flanged, having flanges with

slotted holes. The two types are not interchangeable. A section of nestable pipe has a semicircular shape and is $25\frac{1}{2}$ inches long. It has an effective length after overlap of 24 inches for interior sections, and varies in diameter from 8 to 84 inches. Arch culverts can be fabricated from these sections, with the haunches anchored in a concrete foundation. The foundation is part of the bottom slab of the structure, but the bottom slab may be omitted if suitable rock is encountered. Gage of pipe selected is based on tables IV and VI. Where strutting is indicated (table VI) it shall be accomplished as described in paragraph 17.

16. Assembly of Nestable Corrugated Pipe

a. Tools and Fittings. A box of fittings and tools, some of which are shown in figure 20, accompanies the *notched type* pipe and contains the following: combination draw and pinch bar, curved-back stitches and stitch-bending bar for 15-inch and smaller culverts, straight-back stitches and stitch-bending bar for 18-inch to 30-inch culverts, and hook-and-eye bolts and bolt wrench for 36-inch and larger culverts. The only fittings necessary to fasten the *flange type* sections together are the nuts and bolts accompanying the sections.

b. Checking Sections. Check semicircular diameters of all sections for correct size. Reduce oversize diameters by pressing one edge on ground, or by using a convenient prop or support to apply leverage pressure (fig. 21). Spread undersize diameters by dropping sections squarely on their backs (fig. 22). Inspect notches, or flanges, on all sections for breaks or bends. Use a pinch bar to open any closed notches so that the plain edge of the other section will easily slip into place. Straighten any bent flanges.

c. Assembly Procedure. Distribute one bottom and one top section for each 2 feet of culvert length along the line where the culvert is to be assembled. Begin assembly at the outlet end of the culvert and lap each successive bottom section one-half a corrugation inside the previous bottom section (fig. 23). Similarly, each successive top section is lapped one-half a corrugation inside the previous section.

(1) In assembling the notched type pipe, the notched edge of the bottom section can be either to the right or to the left but, once started, all notched edges of bottom sections must be on the same side. When two bottom sections are in place, center the first top section over the joint of the two bottom sections so that two of the assembly holes in the top will match two of the assembly holes in the bottom sections (fig. 24). Use the combination draw and pinch bar to seat the horizontal joint as shown in figure

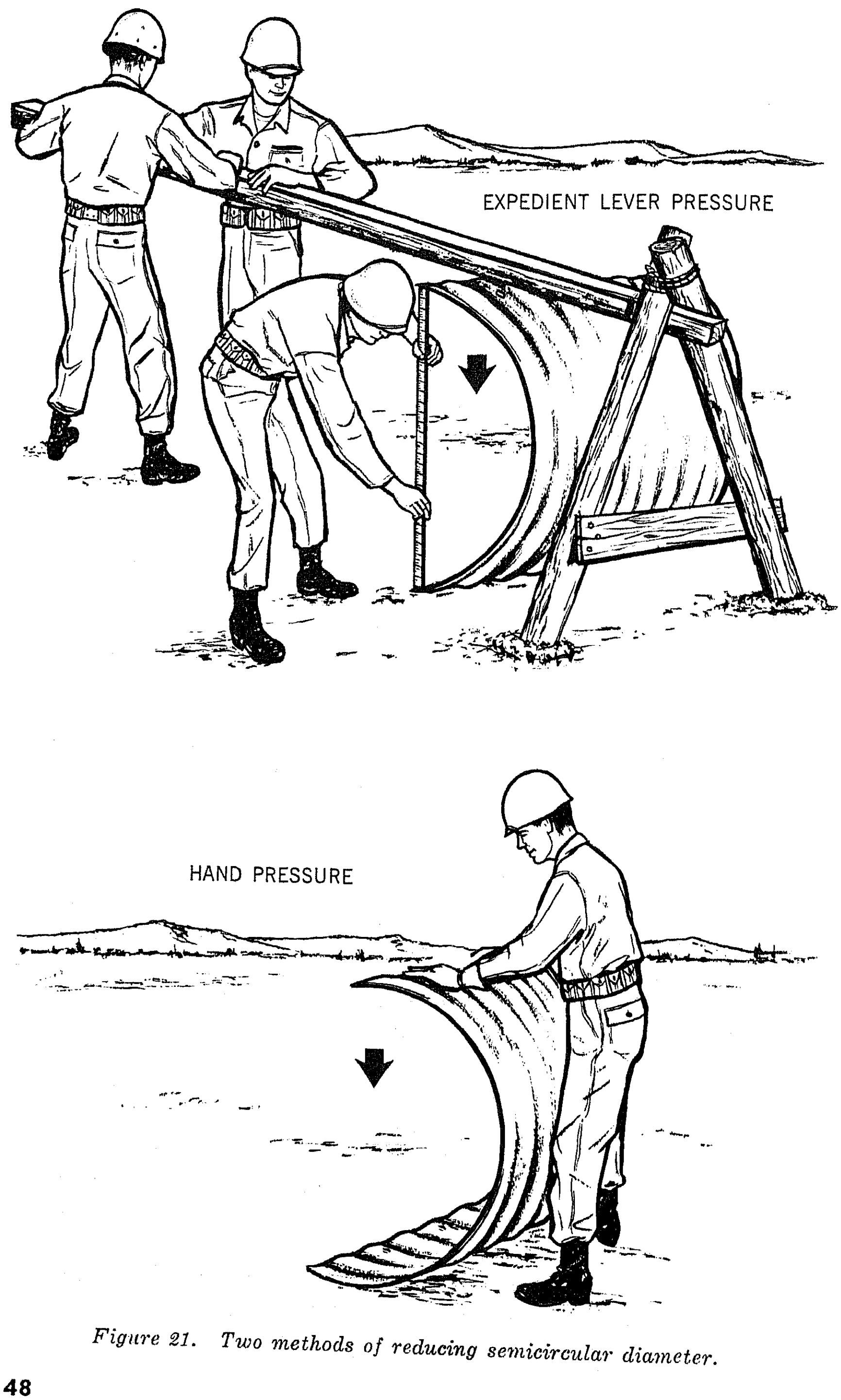


Figure 21. Two methods of reducing semicircular diameter.

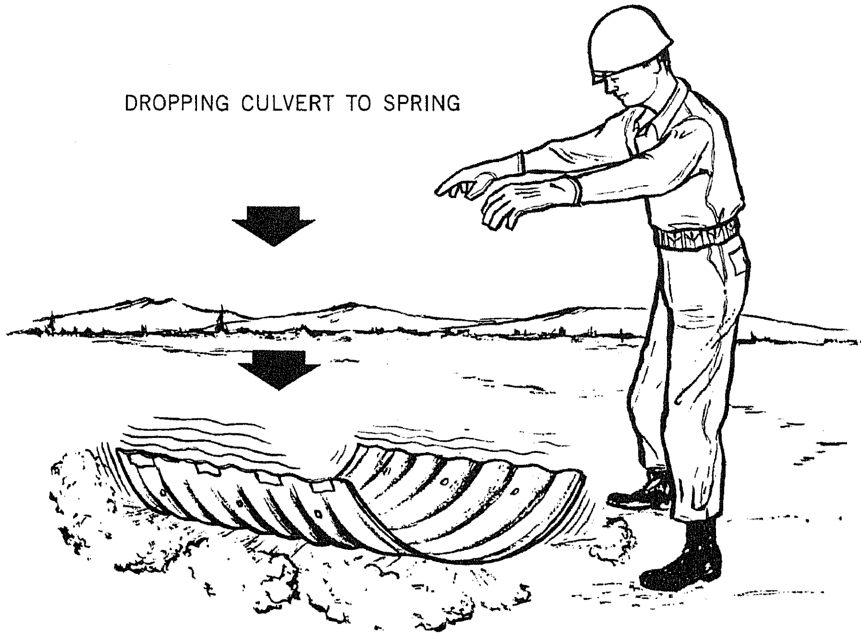


Figure 22. Increasing semicircular diameter.

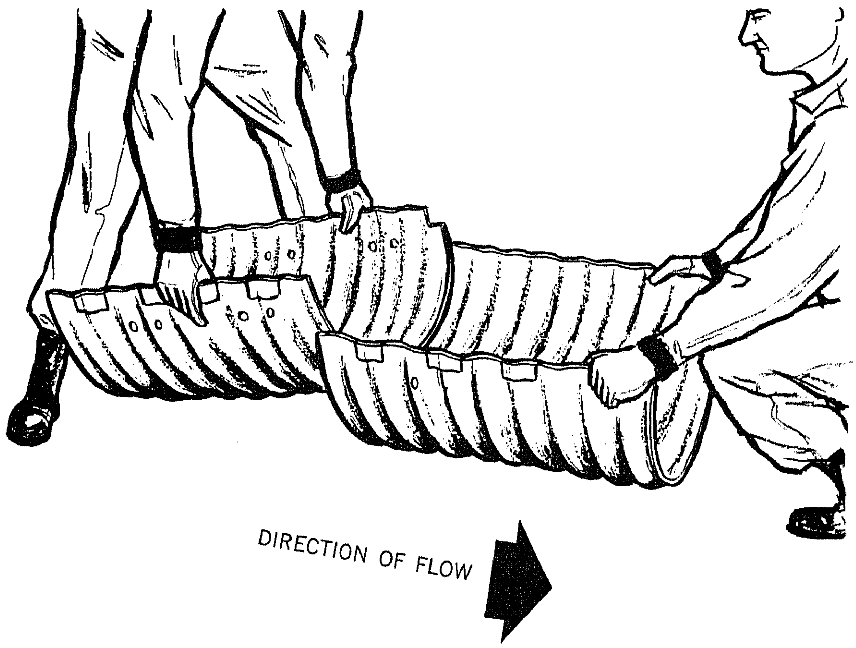
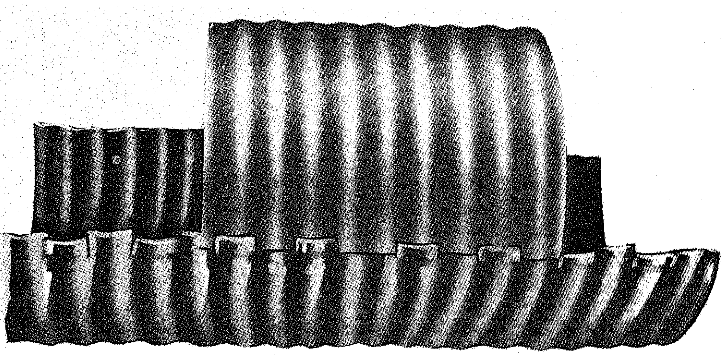
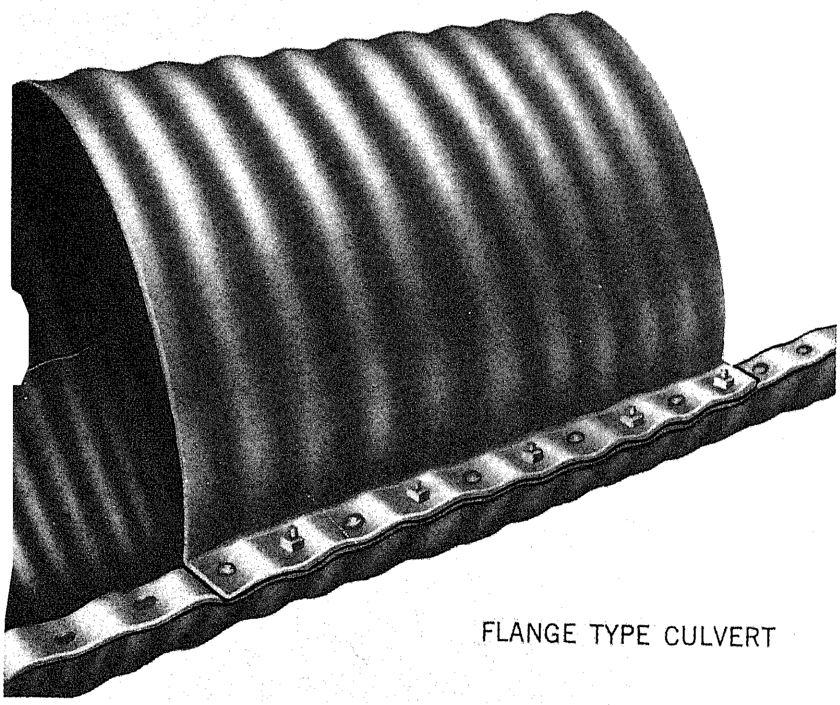


Figure 23. Beginning of assembly. Upstream section is lapped one-half corrugation inside downstream section for notched and flanged pipe.

20. The horizontal joints are secured with either stitches or hook-and-eye bolts; these are also shown in figure 20. Figure 25 shows an assembled section of notched type culvert.

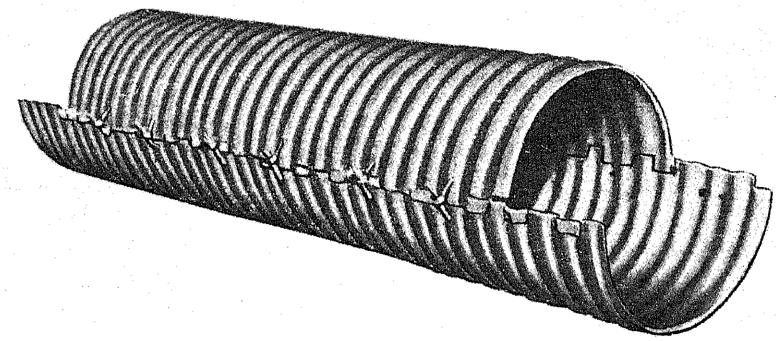


NOTCHED TYPE CULVERT



FLANGE TYPE CULVERT

Figure 24. Matched assembly holes of bottom and top sections.



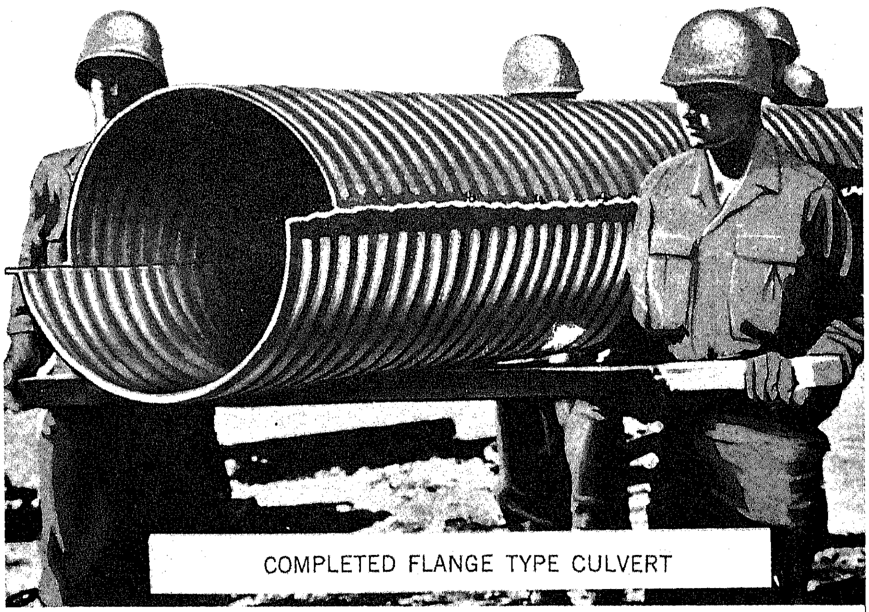


Figure 25. Assembled nestable corrugated-metal culverts.

(2) In assembling the flange-type pipe, note that the flange on one side of each section is punched with holes in the vales, while the flange on the other side has holes in the crests. When laying out the bottom sections, the sections must be placed so the flanges with holes in the crests are

on one side of the culvert, and the flanges with holes in the vales are on the other (fig. 24). Figure 25 shows an assembled flange-type culvert.

17. Strutting Nestable Pipe

a. Basic Procedure. Nestable culvert pipe with diameters and depth of fill requiring vertical elongation as shown in table VI should be strutted before backfill is placed. Strutting is removed after the backfill is completed to allow the pipe to resume its normal shape.

b. Size and Placement of Struts and Sills. The strutting members consist of one lower sill, two upper sills, vertical struts, jack struts, and, usually, compression caps and bearing blocks (fig. 26). All members must be sound timber. Struts and sills must be cut squarely in order to sit level and join evenly. Compression caps must not be less than 10 inches long to provide ample room for both the vertical strut and the jack strut. Length of the vertical strut must be the elongated diameter of the pipe minus the combined thickness of the compression caps and the upper and lower sills. Jack struts are shorter pieces, but long enough to accomplish the required stretching within the lifting range of the jack. All joints in upper and lower sills must be made at the vertical struts. However, the two upper sill members should not be joined on the same vertical strut, but should be installed with joints staggered on alternate struts (fig. 27). Strut spacing is shown in table VII.

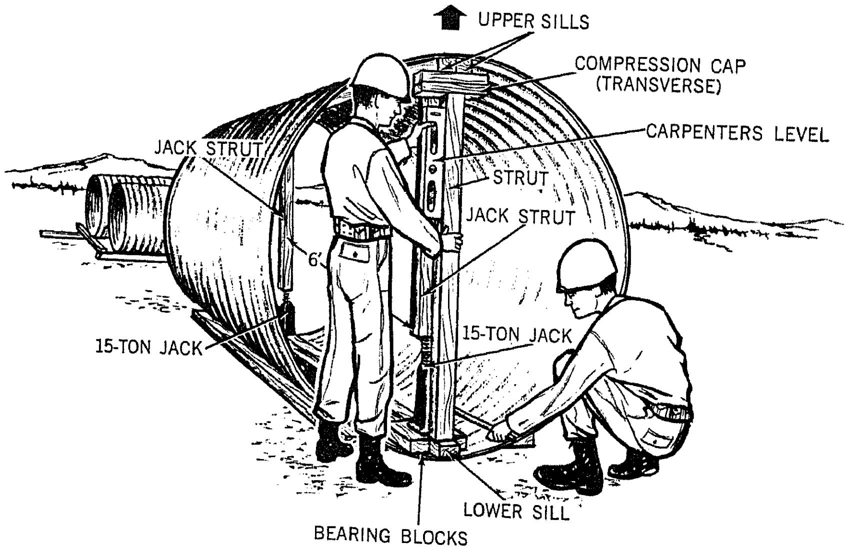


Figure 26. Details of strutting equipment and materials.

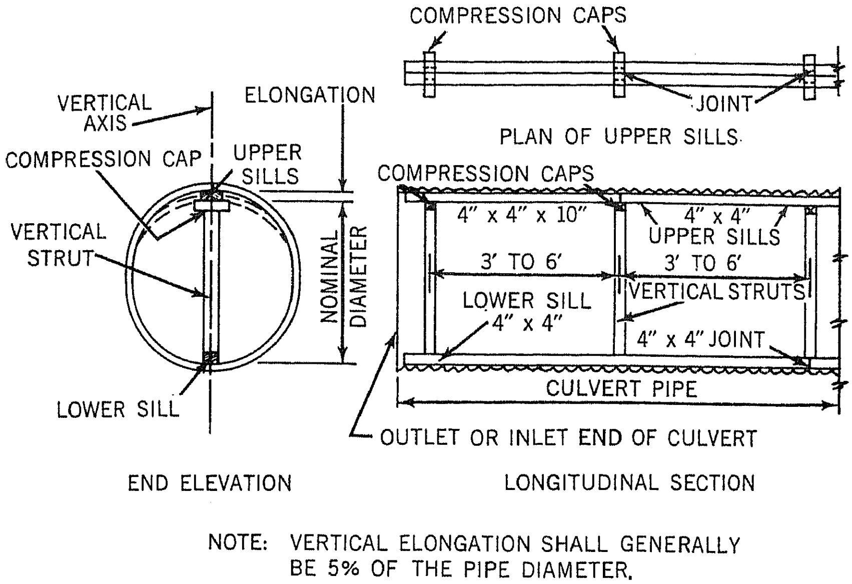


Figure 27. Strutting diagram showing end and longitudinal views.

Table VII. Strut Spacing Using 4- by 4-Inch Timbers With Transverse Compression Caps

Pipe diameter (inches)	Fill heights in feet		
	Up to 20	20-30	30-40
	Spacing of struts in feet		
60	6	6	5
66	6	6	4.5
72	6	5	4
78	6	4.5	3.5
84	6	4	3

c. Installation of Strutting. Not less than four men must work together, proceeding as follows:

(1) Distribute the prepared sills, struts, and other members throughout the entire length of the pipe, where they will be needed.

(2) Lay the lower sills in their proper position, with additional bearing blocks alongside to form bases for the jacks.

(3) Place the first jack on the lower sill just ahead (toward the opposite end of the culvert) of where the first vertical strut will be placed.

(4) Place the second jack just in back of where the second vertical strut will be placed.
(5) Hold the upper sills in place while placing jack struts on the jacks. Use a carpenter's level to plumb all struts.
(6) Place a jack-strut compression cap between the jack strut and the upper sills.
(7) Apply pressure evenly on both jacks.
(8) Insert the vertical strut and compression cap as soon as jack pressure has made enough room for them to fit between the upper and lower sills. Release the jack pressure and remove jack, jack strut, and jack-strut compression cap. Be careful to prevent the jacks from slipping out of place while pressure is being applied, and to keep the vertical struts in alinement when releasing jack pressure.
(9) Remove the first jack and place it just ahead of the position where the third vertical strut will be placed.
(10) Set the third jack strut, jack-strut compression cap, and jack before removing the second jack and setting up the fourth jack strut.
(11) Repeat this procedure for the full length of the culvert.

18. Open-Top and Box Culverts

a. Open-Top Culverts. Open-top culverts, constructed of sawed timbers (fig. 2), logs (fig. 28), or rocks, may be used on steep grades where heavy flow is expected down the road surface. Like ditch-relief culverts (par. 14*g*), they are placed at an angle of about 60° to the centerline or 30° to the perpendicular of the road (fig. 28).

b. Log Box Culverts. Sized-timber or log box culverts are constructed with a square or rectangular cross section. They must be designed to prevent side as well as roof collapse. Typical designs of suitable timber and log culverts are shown in figures 29 to 36. When the soil beneath the culvert has low bearing strength, stringers and sleepers are used as a foundation (fig. 19). Log box culverts can be made several ways. The best method is to run the side logs along the length of the culvert, with the logs which form the top and bottom laid at right angles to them. Spreaders and stakes are placed inside the culvert to provide stability (fig. 29). This culvert may be modified by placing the stakes on the outside of the culvert and extending them flush with the top logs; the top logs then act as spreaders (fig. 30). A third method is to place the side logs perpendicular to the flow line and fit them into notches in the logs forming the top and bottom (fig. 31). This is an excellent culvert, but takes considerable time to build.

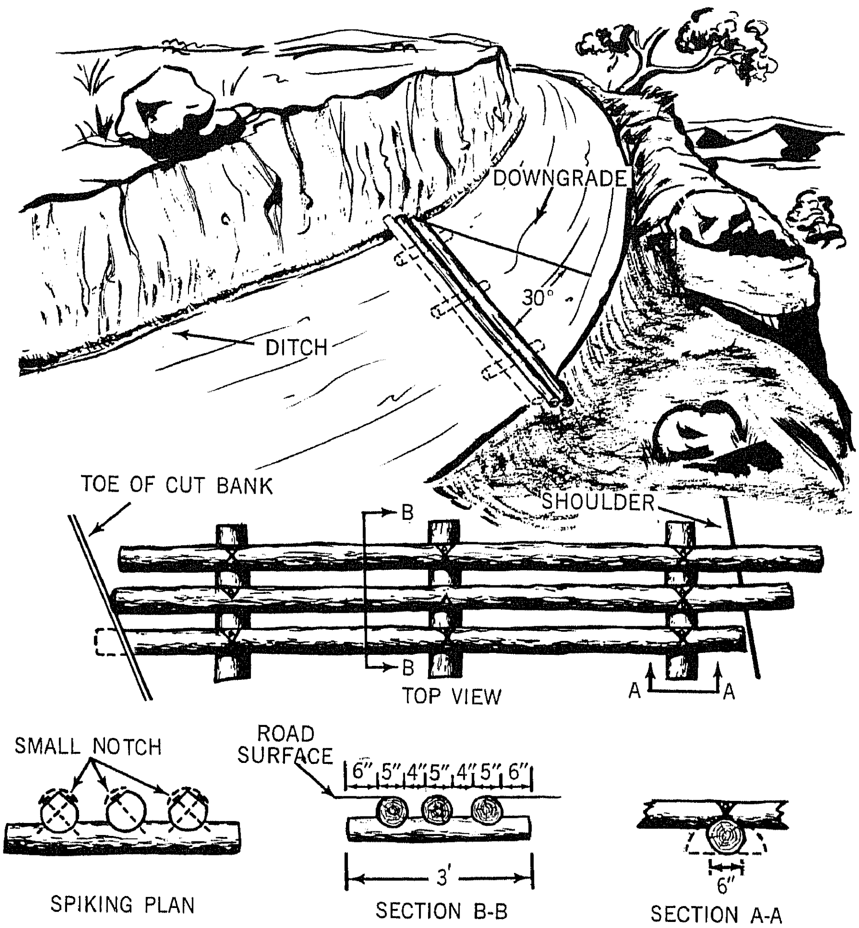


Figure 28. Log open-top culvert.

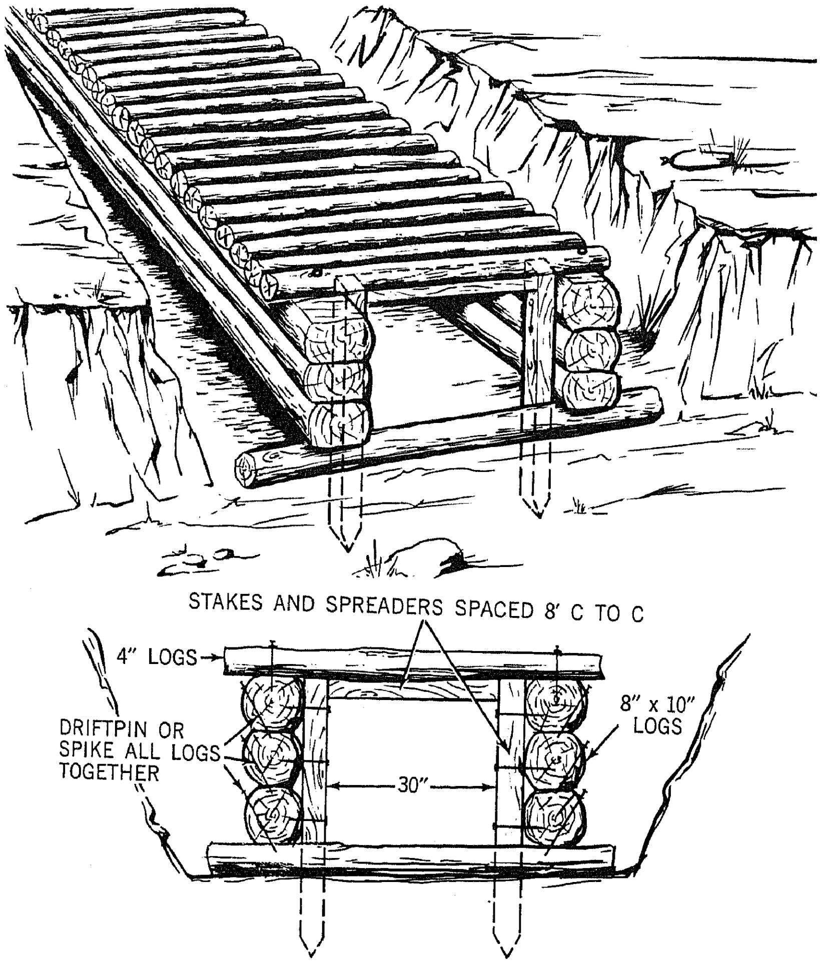


Figure 29. Log culvert, 30-inch.

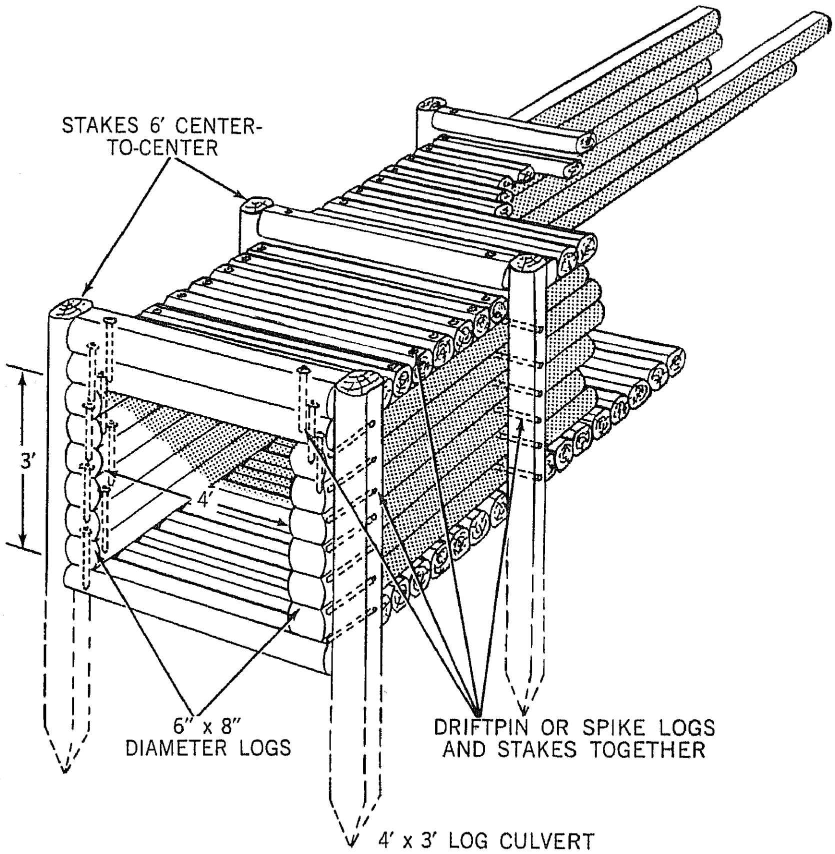


Figure 30. Log culvert, 4- by 3-foot.

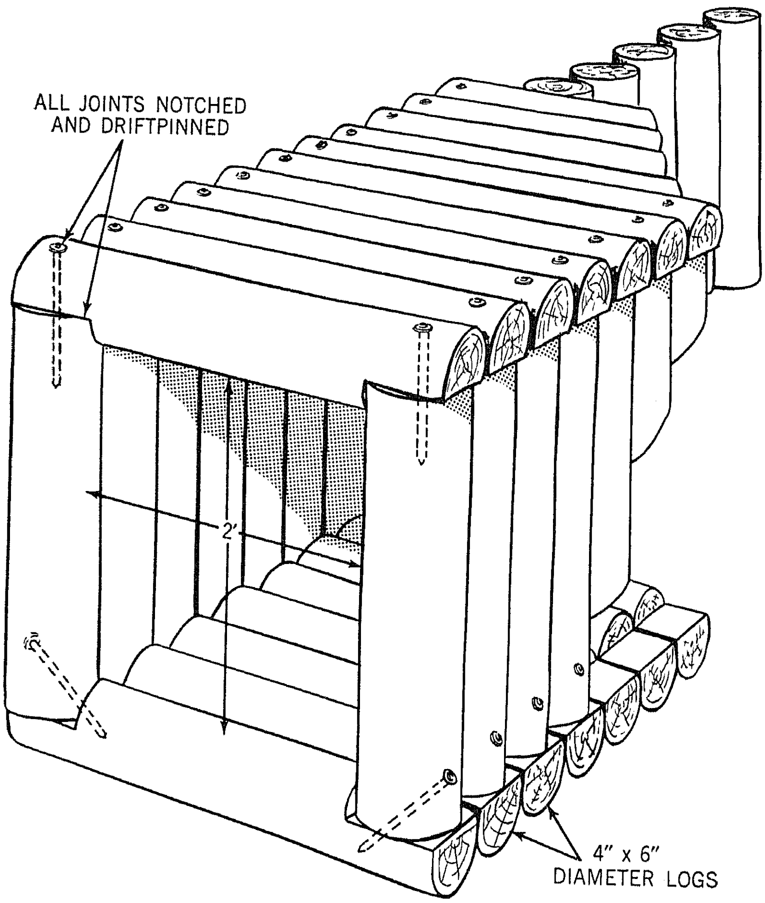


Figure 31. Log culvert, 2- by 2-foot.

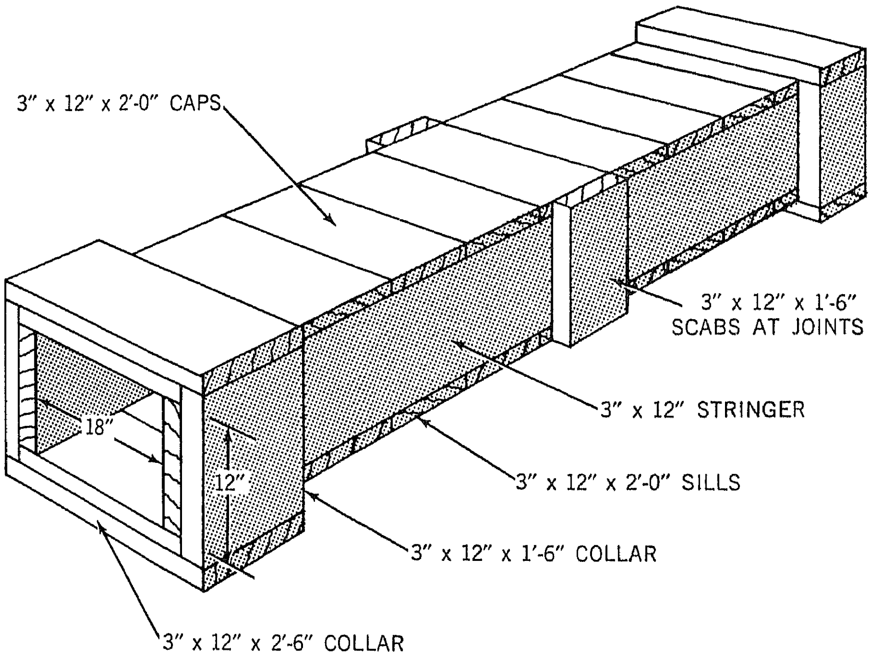


Figure 32. Timber box culvert, 18- by 12-inch.

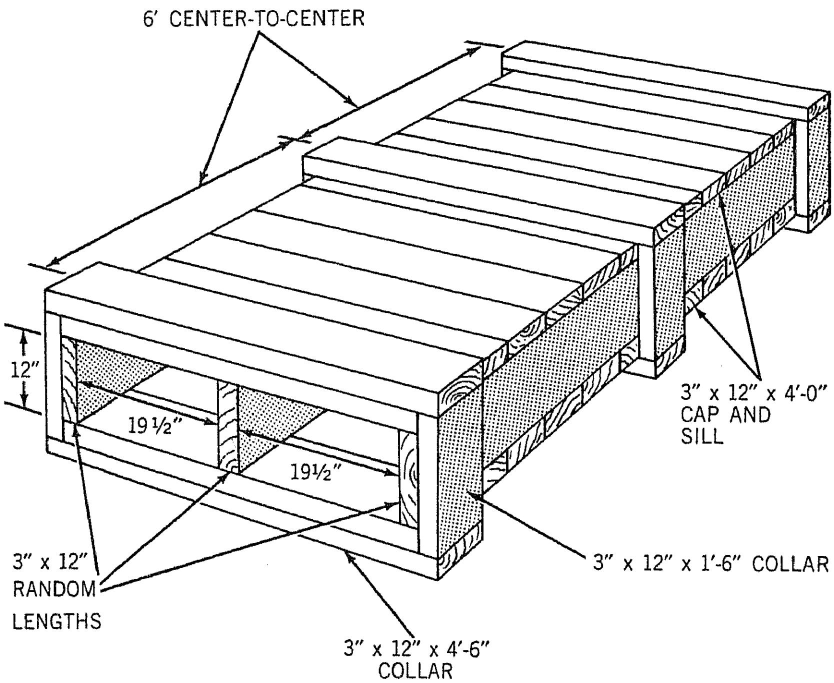


Figure 33. Double box culvert, 19½- by 12-inch.

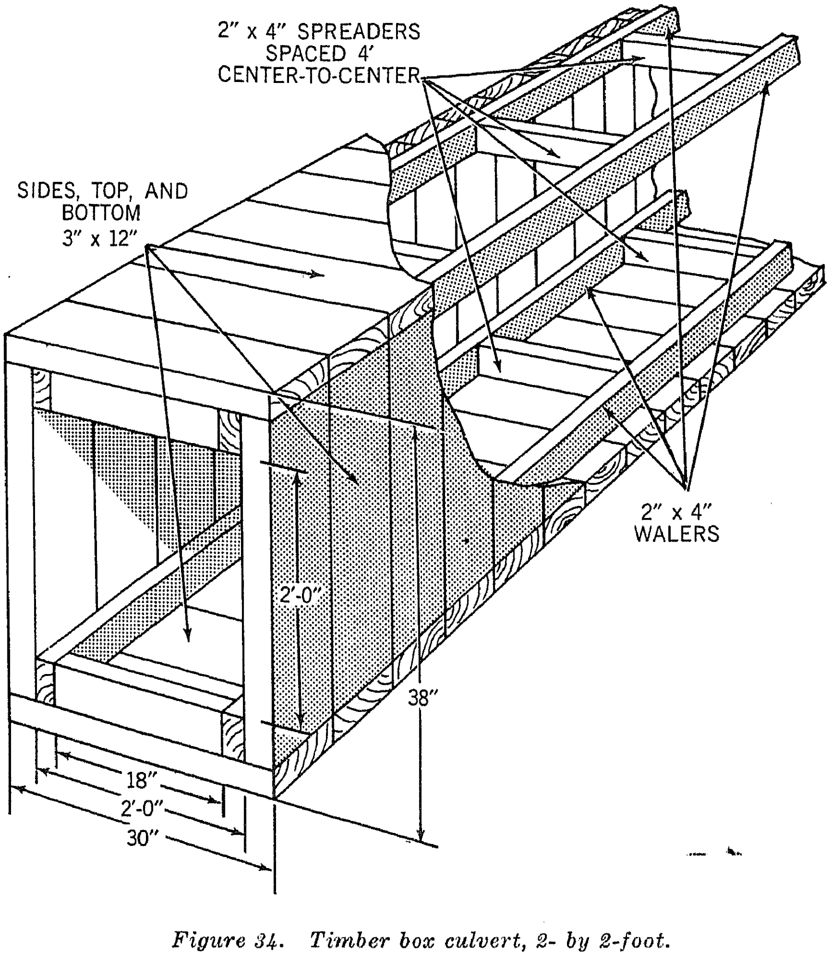


Figure 34. Timber box culvert, 2- by 2-foot.

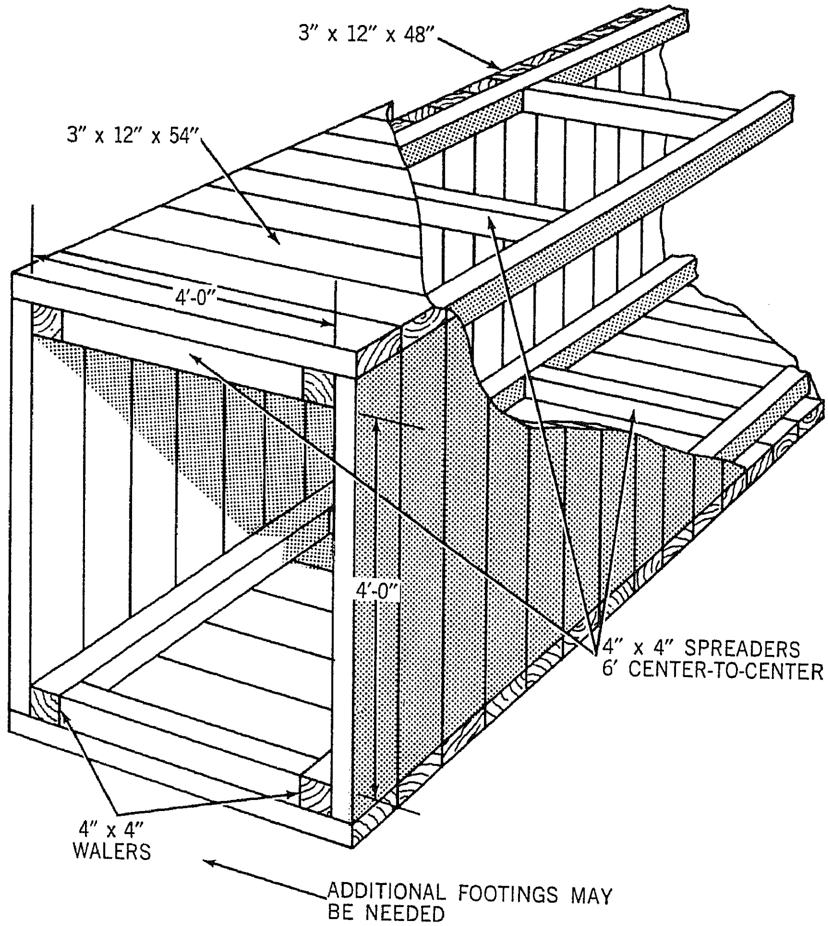


Figure 35. Timber box culvert, 4- by 4-foot.

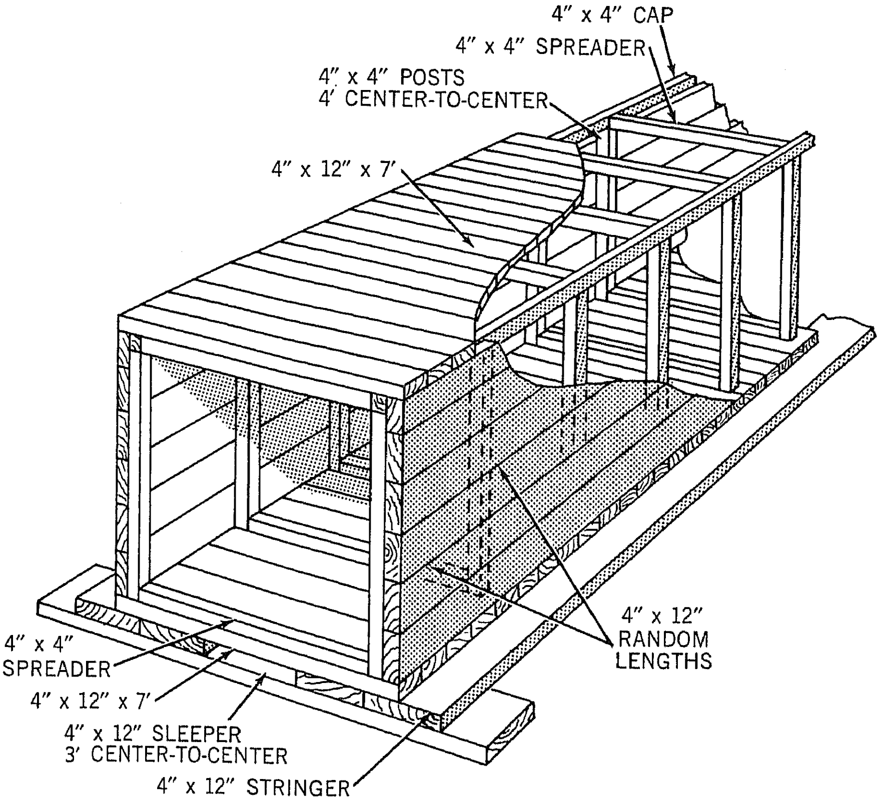


Figure 36. Timber box culvert, 6- by 6-foot.

c. Timber Box Culverts. Timber box culverts can be built with outside bracing or collars (figs. 32 and 33), or with internal bracing (figs. 34–36). Internal bracing should be used whenever possible, since collars do not give the rigidity and strength of internal bracing. Side boards on internally braced culverts are normally placed perpendicular to the flow line to increase the load-bearing strength. Although considerable cutting is required for these culverts, they can be erected rapidly.

d. Concrete Box Culverts. In rear areas, reinforced-concrete box culverts are often required for large drainage areas. In general, the use of a large-diameter pipe or a series of pipes in parallel is preferable to a small concrete box culvert of equal opening because of the saving in both construction time and materials. Either box, circular, or arch concrete culverts are acceptable, although box culverts are usually easier and more economical to construct. Structural design to carry the loads to which they will be subjected is covered in engineer library reference sets.

Figures 37 and 38 show typical sections of a single and a double concrete box culvert with typical placement of reinforcement steel. Concrete culverts are generally built in place and are usually constructed in two sections. First, an invert or base slab is deposited in an earth ditch; then the sides and top are placed in forms. For circular culverts up to about 3 feet in diameter, corrugated-metal pipe may be used as the inside form and left in place. Emptied drums formed into a continuous pipe as described in paragraph 19*a* and *b* may be used for the same purpose. Collapsible wood forms of the type shown in figure 39 are used in sizes up to a 4-foot inside diameter. Rectangular forms of the type shown in figure 40 are used in sizes up to about 10 feet.

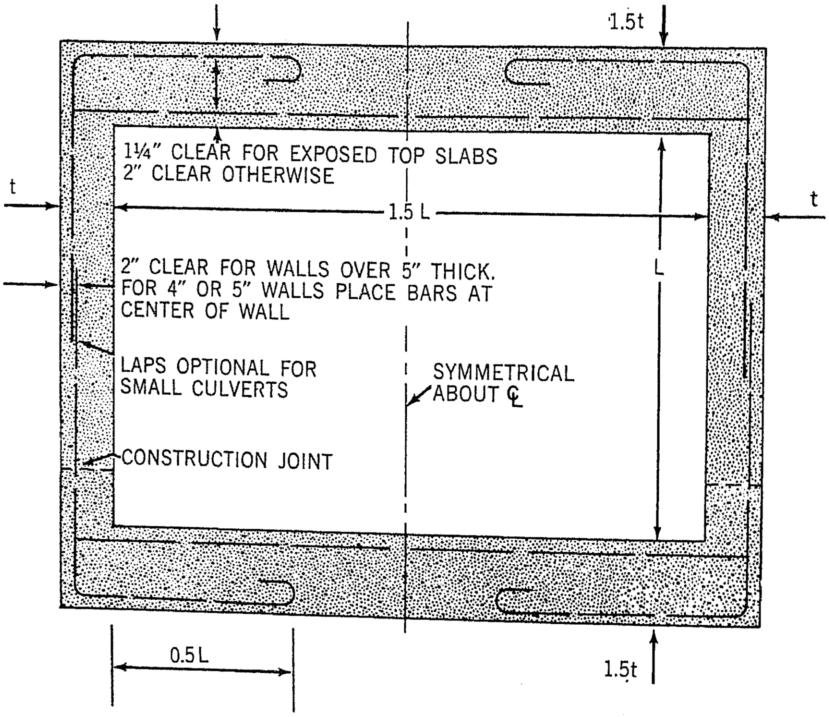


Figure 37. Typical cross section of a single concrete box culvert.

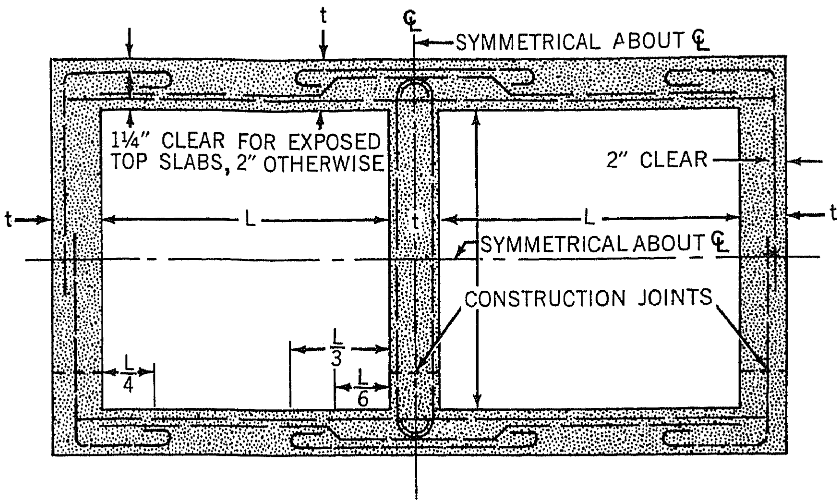


Figure 38. Typical cross section of a double concrete box culvert.

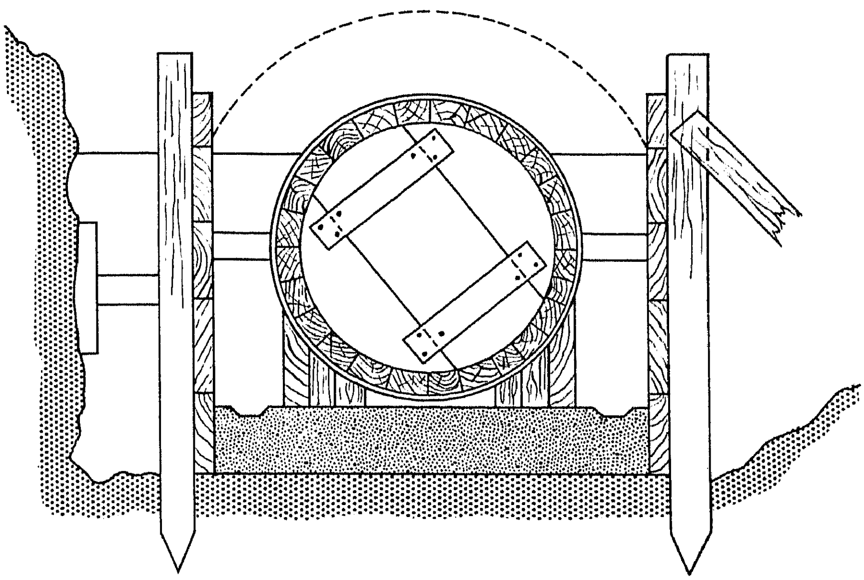


Figure 39. Collapsible concrete-culvert form.

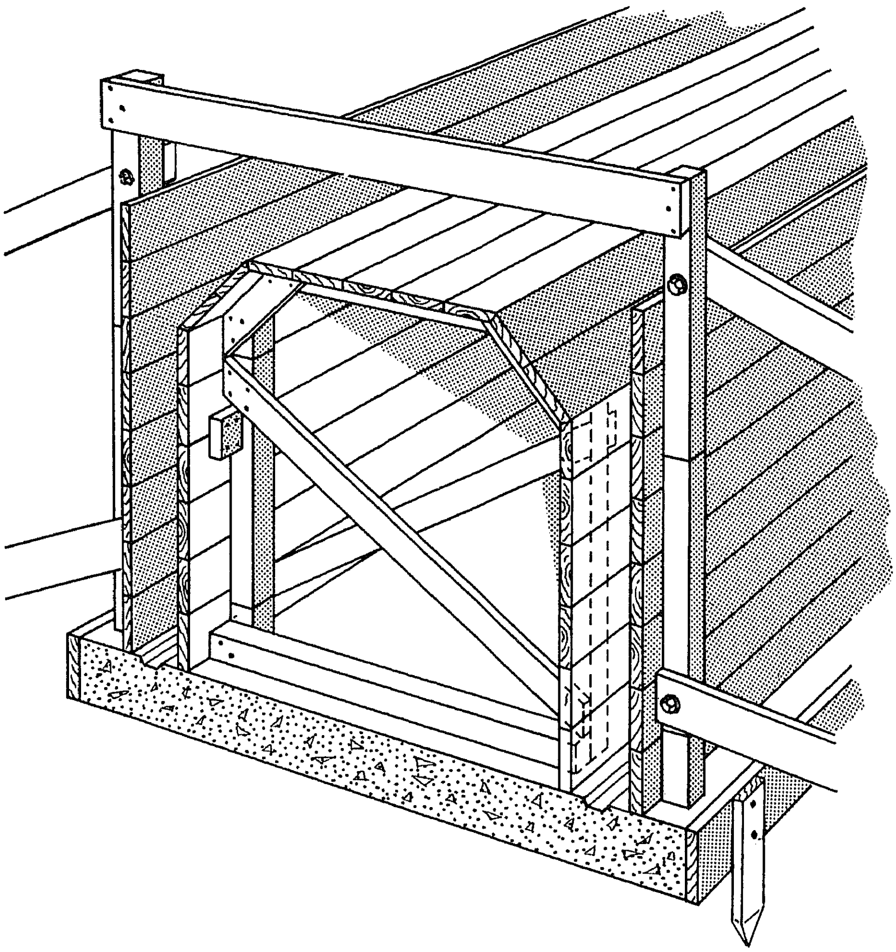


Figure 40. Rectangular concrete-culvert form.

19. Expedient Culverts

Although the resulting structures are not as strong as corrugated-pipe culverts, one of the most useful expedients for forming drainage structures is to use empty oil, gasoline, or asphalt drums. Another is to build box-like culverts from pierced steel plank (PSP) and sandbags.

a. Drum ends can be removed by several methods: three strands of detonating cord may be laid around the rim of the head and then fired; sharp handtools can be used; the pneumatic metal drum opener does an excellent job.

Caution: **Do not use a torch or other tools on gasoline or oil drums unless the drums have been completely emptied. Gasoline and fuel-oil fumes remain long after liquids have been drained, and there is danger of explosion unless the fumes are removed. One method of removing fumes is to blow them out with compressed air. Another is to fill the drums to overflowing with water.**

b. After the ends are removed, a continuous pipe is formed by tackwelding, bolting, or wiring the drums together (fig. 41). To avoid collapse under earth cover and traffic loads, place the entire culvert in as narrow a trench as possible. A cradle of wood or concrete will usually be required to provide an adequate foundation. A minimum of 3 feet of cover should be used. Where roads initially constructed for light loads may later be improved for heavier loads, install drum culverts capable of withstanding the heavier loads. The load capacity can also be increased if a load-distributing layer of logs covered with a minimum of 12 inches of earth is placed over the drums. When possible these logs should be placed perpendicular to the centerline of the culvert with their ends bearing on undisturbed earth.

c. Another type of expedient culvert is easily constructed with sandbags and PSP, as shown in figure 42. Note that the top layer of PSP should be covered with empty sandbags, burlap, or similar material to prevent covering soil from entering the holes in the PSP.

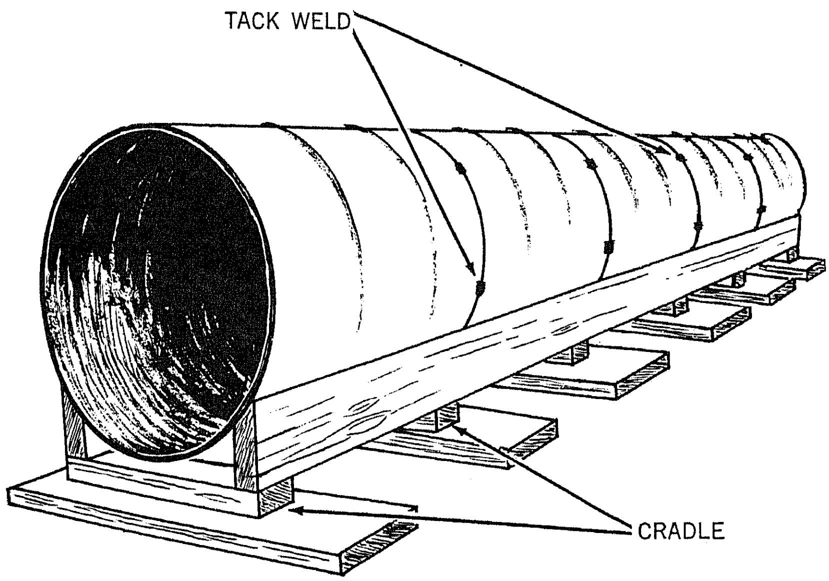


Figure 41. Expedient culvert made from emptied drums.

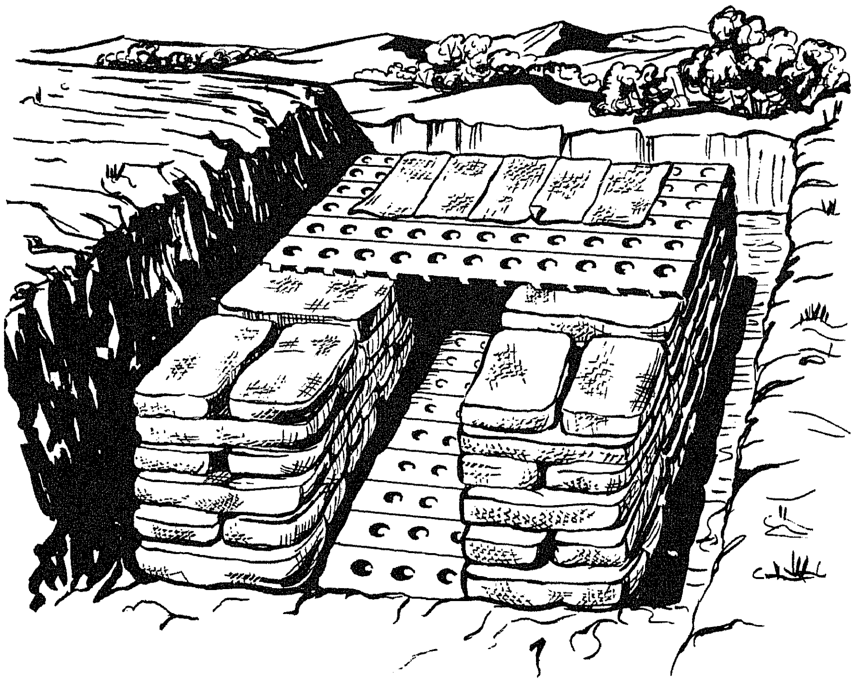


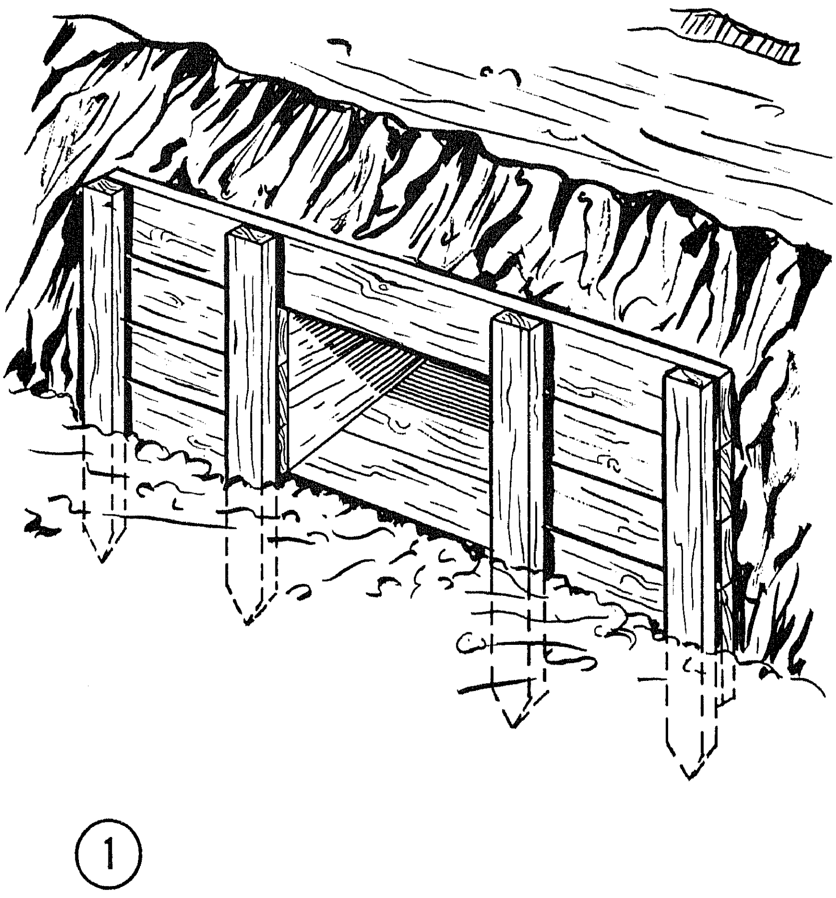
Figure 42. Expedient culvert made from sandbags, PSP, and sacking.

20. Headwalls, Wingwalls, Drop Inlets, and Gratings

a. Headwalls and Wingwalls. Headwalls and wingwalls are constructed to prevent or control erosion, guide water into the culvert, reduce seepage, and hold the ends of the culvert in place. These structures are expensive both in time and materials. Consequently, on the inlet end, the pipe culvert should be extended so that the minimum height and length of headwall are required. As a general rule, headwalls can be omitted on the outlet end of pipe culverts except on steep grades, where they are used to hold culvert sections in place. Headwalls should not protrude above shoulder grade and should extend at least 2 feet outside the shoulder, so they will not be a traffic hazard. If headwalls or wingwalls are not used, the culvert will have to be extended to at least 2 feet beyond the toe of the fill. Headwalls and wingwalls should ordinarily be constructed of materials as durable as the culvert, but sandbags or rubble may be used for temporary installations. Log, timber, or rubble headwalls or wingwalls may be used, as shown in figure 43. For walls less than 5 feet high, the wall may be made of 2- or 3-inch by 12-inch timbers supported by timber piles or posts. Timber sheet piles may be used for head-

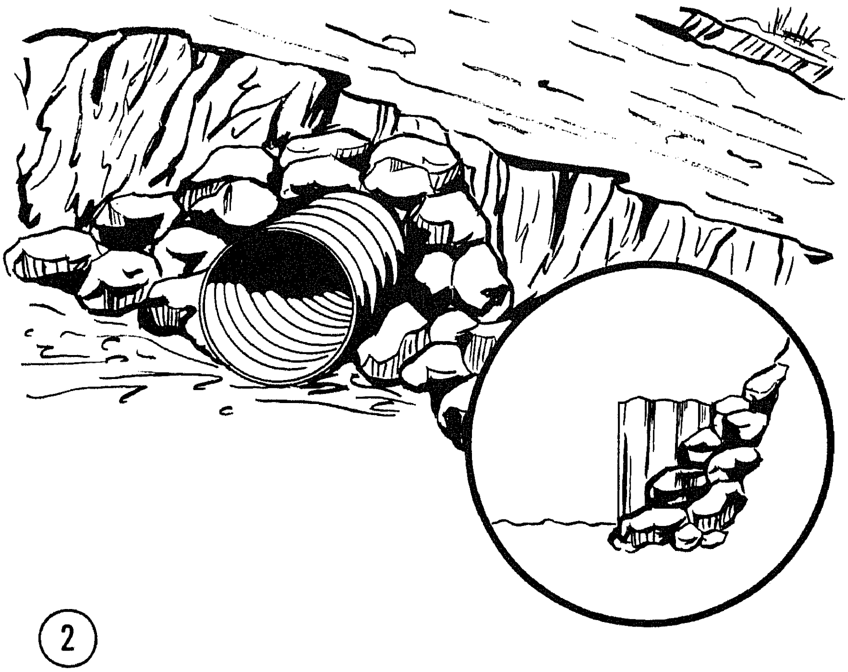
walls and wingwalls; they are described in detail in TM 5-258. Where wingwalls are required to channelize water and prevent the washing out of headwalls, they should be built to fit site conditions. Their height should be sufficient to prevent spilling of embankment material into the waterway. Their top thickness should be the same as the top thickness of headwall to which they are attached. The embankment side is battered 2 inches in 12 inches, but the outside faces are kept plumb.

b. Drop Inlets. A drop inlet (fig. 44) is a vertical culvert or storm-drain entrance that has many uses. A drop inlet may be used to lower the elevation of the culvert entrance below ditch elevation where fill does not provide sufficient cover or where discharge velocity is erosive and cannot be controlled by the use

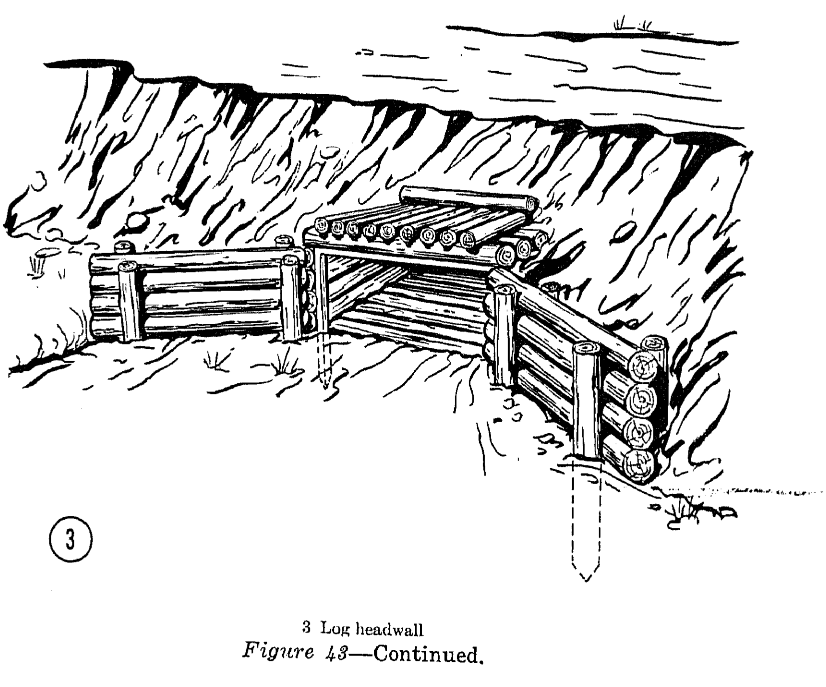


1 Plank headwall

Figure 43. Headwalls for culverts.



2 Rubble headwall



3 Log headwall

Figure 43—Continued.

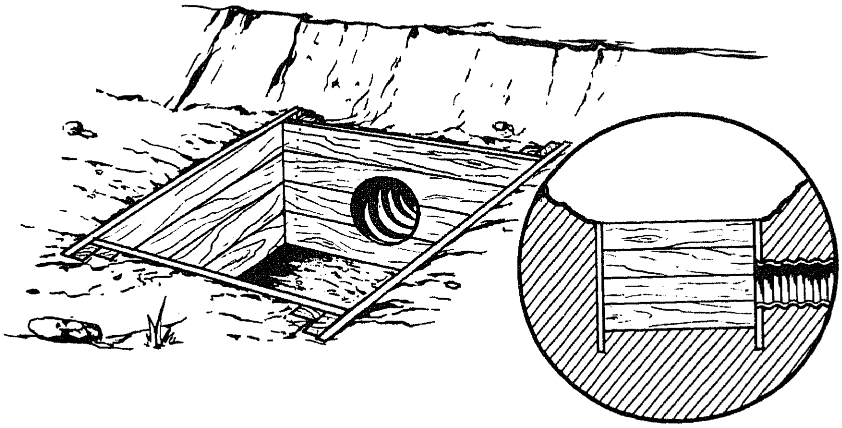


Figure 44. Drop inlet.

of smaller pipes. For storm drains (underground conduits designed to carry surface runoff), drop inlets are used to collect surface water from paved and turfed areas, street gutters, and ditches.

c. Gratings. A grating is a framework of bars or a perforated plate which allows the storm runoff to enter the drop inlet. The grating serves as a sieve or strainer and as a cover for the inlet.

d. Drop Inlet and Grating Criteria.

(1) Drop inlets may be constructed of concrete, brick, wood, or pipe.

(2) Inlet gratings should be fabricated of steel bars, steel plate, cast iron, or reinforced concrete with adequate strength to withstand anticipated loading.

(3) Inlet gratings should be placed 0.2 foot below grade to make allowance for settlement, provide a sumped area, and insure positive interception of surface runoff.

(4) The area of the grate openings should be estimated as 50 percent of the total grate area.

(5) Grate openings should generally be at least 18 inches long and placed parallel to the direction of flow.

(6) A safety factor of 50 percent (1.5 x total grate area) for paved areas and 100 percent (2.0 x total grate area) for turfed areas should be used to compensate for debris caught in the grating.

e. Determination of Grate Size.

(1) Determine the rate of runoff (TM 5–330) from the area

which drains into the drop inlet. This will be the required discharge capacity (Q) of the grate.

(2) Determine the head (H) or depth of water on the grate at the time of peak runoff. When a drop inlet is used in a ditch, the head will be the depth of water running in the ditch.

(3) With the head (H) and the design discharge capacity (Q), use table VIII to select the minimum size of grating required.

(4) Multiply the indicated grating size by the appropriate safety factor to determine the actual grate size to be used.

21. Culvert Hydraulic Design Principles

The discharge capacity (Q) of a culvert is the amount of water which the culvert will carry in a unit of time, usually expressed in cubic feet per second. For a particular culvert of known size, shape, and interior roughness (Manning's coefficient of roughness, n), the discharge capacity of the culvert is controlled by one or more of the following factors: the elevation of the water at the culvert inlet; the hydraulic gradient (S) of the culvert; the length (L) of the culvert; the elevation of the tailwater at the culvert outlet; and the type of inlet. Except for drop inlets, the type of inlet is not generally considered in military culvert design, but it should be remembered that the discharge capacity of a culvert will be increased, particularly in short culverts on steep slopes, by a smooth-transition type of inlet.

a. Culvert Inlet Water Elevation. For roads, airfields, and railroads where extensive ponding at the inlets of culverts is not desirable, culverts are usually designed to utilize the smallest size and least number of available pipes that have a total discharge capacity sufficient to pass the peak runoff from the design storm without allowing the water surface at the inlet to become higher than the top of the inlet. For areas in which temporary ponding has been deliberately planned, culverts are designed to utilize the minimum culvert materials. For methods of calculating runoff and for ponding design see TM 5–330.

b. Culvert Hydraulic Gradient. The hydraulic gradient (S) of a culvert is one of the more important controls of culvert discharge capacity. For culverts that are designed to avoid ponding and are unsubmerged at the inlet and outlet, the hydraulic gradient can be assumed equal to the longitudinal slope of the culvert. The hydraulic gradient for any culvert can be satisfactorily estimated as the slope in feet per foot, calculated by dividing the head (H) on a culvert by the length (L) of the culvert ($S = H/L$). Head is the difference in elevation between: (1) each end of a culvert if the inlet and outlet are not submerged; (2) the water surface

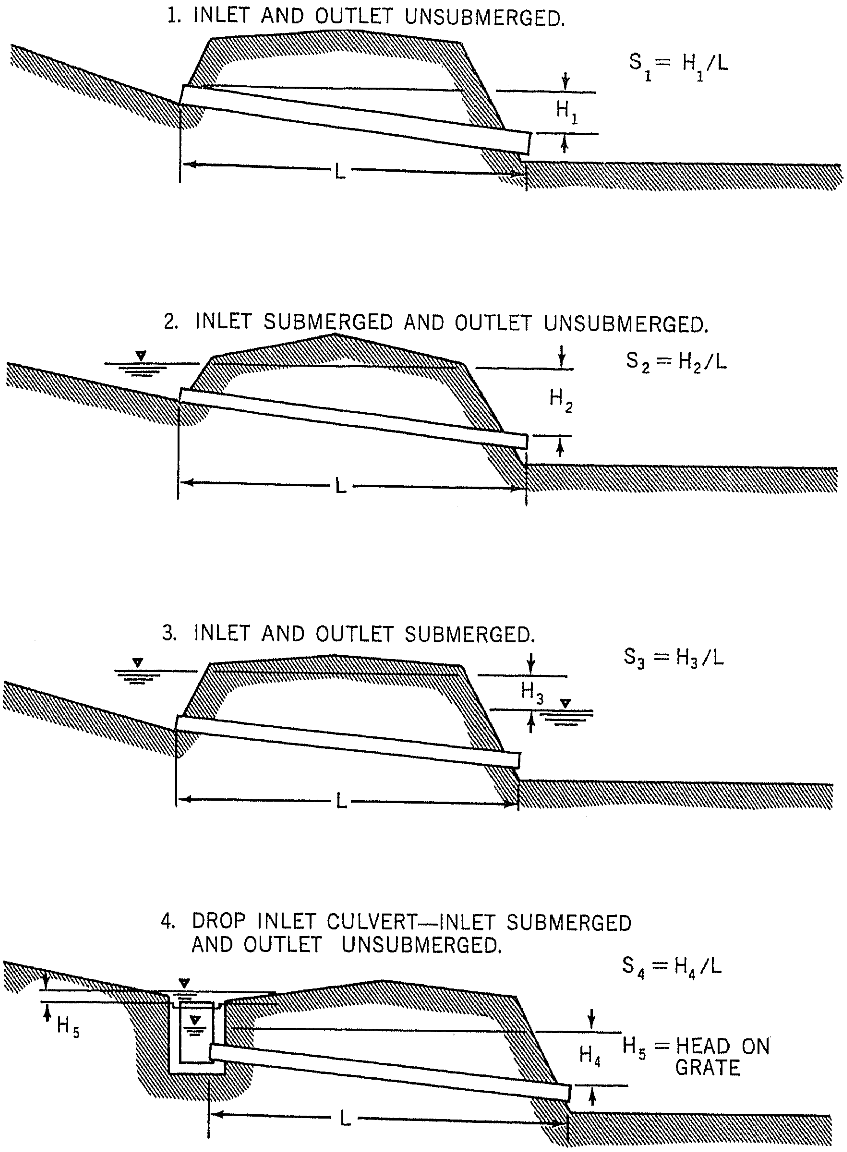


Figure 45. Culvert hydraulic gradient (S) and head (H).

directly above the inlet and the top of the outlet if the inlet is submerged and the outlet is not submerged; and (3) the water surface directly above the inlet and the water surface directly above the outlet if both the inlet and outlet are submerged. The head and hydraulic gradient are illustrated in figure 45.

Table VIII. Discharge Capacity of Square Grate Inlets in Cubic Feet Per Second

Grate size, inches	Grate opening, sq. ft.	Head of water on grate, feet																	
		0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
6x6	0.12	0.3	0.4	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.2	1.3	1.5	1.6	1.8	1.9	2.0	2.1
9x9	0.28	0.7	1.0	1.2	1.4	1.5	1.7	1.8	1.9	2.0	2.1	2.6	3.0	3.4	3.7	4.0	4.3	4.5	4.8
12x12	0.50	1.1	1.7	2.1	2.4	2.7	2.9	3.2	3.4	3.6	3.8	4.7	5.4	6.0	6.6	7.1	7.6	8.1	8.5
15x15	0.78	1.3	2.7	3.2	3.8	4.2	4.6	5.0	5.3	5.6	5.9	7.3	8.4	9.4	10.3	11.1	11.9	12.6	13.3
18x18	1.12	1.6	3.8	4.7	5.4	6.0	6.6	7.1	7.6	8.1	8.5	10.5	12.1	13.5	14.8	16.0	17.1	18.1	19.1
21x21	1.53	1.9	5.2	6.4	7.4	8.2	9.0	9.7	10.4	11.0	11.6	14.2	16.4	18.4	20.1	21.8	23.3	24.7	26.0
24x24	2.00	2.2	6.1	8.3	9.6	10.7	11.8	12.7	13.6	14.4	15.2	18.6	21.5	24.0	26.3	28.4	30.4	32.2	34.0
30x30	3.12	2.7	7.6	13.0	15.0	16.8	18.4	19.9	21.2	22.5	23.7	29.1	33.6	37.5	41.1	44.4	47.5	50.3	53.1
36x36	4.50	3.2	9.1	16.7	21.6	24.7	26.5	28.6	30.6	32.4	34.2	41.9	48.3	54.0	59.2	63.9	68.3	72.5	76.4
42x42	6.12	3.8	10.6	19.5	29.4	32.9	36.0	38.9	41.6	44.1	46.5	57.0	65.9	73.4	80.9	87.0	93.0	98.7	104.0
48x48	8.00	4.3	12.1	22.3	34.3	43.0	47.1	50.8	54.3	56.6	60.8	74.4	85.9	96.0	105.2	113.7	121.5	128.9	135.9

Notes[3]

1. Values to left of HEAVY line were calculated from the weir formula: $Q = 3LH^{3/2}$, L = Perimeter
2. Values to right of HEAVY line were calculated from the orifice formula: $Q = 5.37AH^{1/2}$, A = Grate Opening
3. Clear opening between grate bars was taken to be 50% of total grate area.
4. Grate size should be increased 50% in paved areas.
5. Grate size should be increased 100% in turfed areas.

c. Critical Slope. For a given size of culvert and a given head (H) on the culvert, the discharge capacity of a culvert will increase as the hydraulic gradient increases until the hydraulic gradient becomes equal to or greater than a "critical" slope (S_c). As the hydraulic gradient is increased beyond "critical," the discharge of the culvert remains constant but the area decreases since the pipe does not flow full, and the velocity of flow in the culvert necessarily increases as may be seen from $V = Q/A$, where A equals the cross-sectional area of waterflow. Critical slope is the minimum slope of the hydraulic gradient that will permit maximum discharge.

d. Design of Box Culverts. The characteristics of flow through square or rectangular culverts are different from those of flow through round culverts, even with the same slope, culvert lining, inlet, and outlet. However, the difference in capacity relative to cross-sectional area is negligible. Box culvert sizes can be determined by computing the cross-sectional area required for a pipe and designing a box culvert of the same material as the pipe and with the same cross-sectional area.

e. Design of Pipe Culvert With Unsubmerged Inlet.

(1) Determine the rate of runoff (TM 5–330) that the culvert must drain. This will be the required capacity (Q_r) of the culvert.

(2) Determine the maximum permissible discharge velocity (V_{max}) from table IX for the outfall to which the culvert will discharge. See paragraph 33 for identification of soil symbols.

(3) Draw a cross section of the fill or embankment showing the elevations of inlet and outlet. Compute the culvert slope (S), and side or embankment slope (S/S).

(4) Determine sizes and gages of corrugated-metal pipe (CMP) or sizes and strengths of concrete pipe available.

(5) Calculate the depth of fill at the outside edge of the shoulder. Consider only pipe sizes for which adequate cover can be provided within the established grade limitations.

(6) From table X or XI, as appropriate, determine the pipe capacity (Q_p). Divide the required capacity of the culvert (Q_r) by the pipe capacity (Q_p) to find the number of pipes required. If a fraction results use the next larger whole number.

Table IX. Maximum Flow Permissible in Different Types of Open Channels

Ditch Lining	$V_{max} fps$
1. Natural Earth	
a. Without Vegetation:	
(1) Rock	
(*a*) Smooth and uniform	20
(*b*) Jagged and irregular	15–18
(2) Soils	

Open Channels	Coarse grained	Gravel and Gravelly soils	GW		6–7
			GP		7–8
			GM	d	3–5
				u	2–4
			GC		5–7
		Sand and Sandy soils	SW		1–2
			SP		1–2
			SM	d	2–3
				u	2–3
	Fine grained	Silts and clays, LL < 50	SC		3–4
			CL		2–3
			ML		3–4
			OL		2–3
		Silts and clays, LL > 50	CH		2–3
			MH		3–5
			OH		2–3
	Highly organic		PT		2–3

b. With Vegetation:	$V_{max} fps$
(1) Average turf	
(*a*) Erosion resistant soil	4–5
(*b*) Easily eroded soil	3–4
(2) Dense turf	
(*a*) Erosion resistant soil	6–8
(*b*) Easily eroded soil	5–6
(3) Clean bottom with bushes on sides	4–5
(4) Channel with tree stumps	
(*a*) No sprouts	5–7
(*b*) With sprouts	6–8
(5) Dense weeds	5–6
(6) Dense brush	4–5
(7) Dense willows	8–9

Table IX. Maximum Flow Permissible in Different Types of Open Channels—Continued

2. Paved

	$V_{max}fps$
a. Concrete, w/All Surfaces:	
(1) Trowel finish	20
(2) Float finish	20
(3) Formed, no finish	20
b. Concrete Bottom, Float Finished w/Sides of:	
(1) Dressed stone in mortar	18–20
(2) Random stone in mortar	17–19
(3) Dressed stone or smooth concrete rubble	15
(4) Rubble or random stone	15
c. Gravel Bottom, Sides of:	
(1) Formed concrete	10
(2) Random stone in mortar	8–10
(3) Random stone or rubble	8–10
d. Brick	10
e. Asphalt	18–20

(7) Determine the length of the culvert from the cross section, graphically, or calculate the length as illustrated in *f* and *g* below.

f. Example, Unsubmerged Inlet. A culvert is required at station 10 + 50, a cross section of which is shown in figure 46. The inlet invert elevation is 594.0 feet, the shoulder elevation is 598.6 feet, culvert slope is 1.5 percent, the Q that the culvert must discharge in a GC soil is 28 cfs, the maximum live load on the culvert will be a 50,000 pound F101 aircraft. The following culvert materials are available: 14 gage 12″, 18″, 24″, 30″, and 36″ plain CMP.

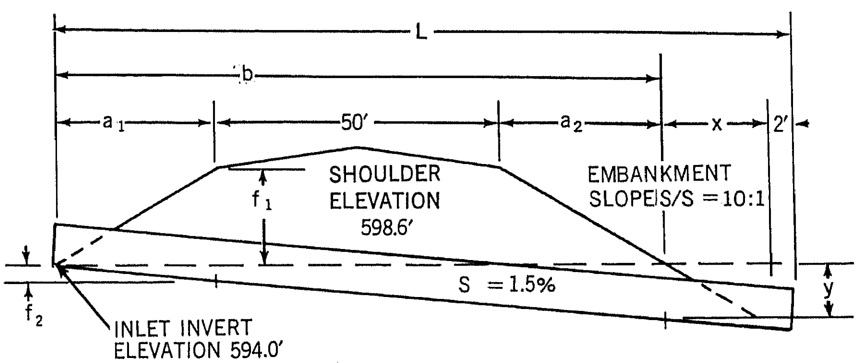


Figure 46. Cross section at station 10 + 50.

Table X. Capacity of Culverts With Free Outlet, CMP

With water surface at inlet same elevation as top of pipe, and outlet unsubmerged.
Values are in cubic feet per second. Manning's n = .024

S in %	Diameter of pipe, in inches										
	V_{fps}	12″	18″	24″	30″	36″	42″	48″	60″	72″	V_{fps}
0.4	2	1.3	3.9	8.4	15	25	37	53	96	160	7
0.6		1.6	4.8	10	18	30	45	64	120	190	8
0.8	3	1.8	5.4	12	21	34	50	72	130	210	9
1.0		2.0	5.9	13	22	36	54	77	140	220	10
1.2		2.2	6.4	13	24	38	57	80	150	230	11
1.4		2.3	6.6	14	25	39	59	82	150	230	
	4										
1.6		2.4	6.8	14	25	40	59	83	150	230	
1.8		2.5	7.0	14	26	40	59	83			
2.0		2.5	7.0	15	26	40					
2.2		2.6	7.1	15							
	5										
2.4		2.6	7.1								
2.6		2.6									
2.8	6	2.6									

Notes.
1. Underlined values for Q indicate critical slope (S_c) for the respective pipe; slopes steeper than S_c do not result in increased discharge.
2. The stairs indicate velocity of discharge in fps.

Table XI. Capacity of Culverts With Free Outlet, CMP (25 Percent Paved)

With water surface at inlet same elevation as top of pipe, and outlet unsubmerged.
Values are in cubic feet per second. Manning's $n = 0.021$.

S in %	V_{fps}	Diameter of pipe, in inches								
		12″	18″	24″	30″	36″	42″	48″	60″	72″
0.2	2	1.0	3.1	6.8	12	19	30	42	77	130
0.4		1.5	4.4	9.5	17	28	42	62	110	180
0.6		1.9	5.4	11	21	33	50	72	130	210
0.8	3	2.1	6.1	13	23	37	55	77	140	220
1.0		2.3	6.5	14	24	39	57	80	145	230
1.2		2.4	6.8	15	25	40	59	82	150	
1.4	4	2.5	7.0	16	26					
1.6		2.6	7.1							
1.8	5	2.6	7.1							
V_{fps}			6		7		8		9	10

Notes.

1. Underlined values for Q indicate critical slope (S_c) for the respective pipe; slopes steeper than S_c do not result in increased discharge.
2. The stairs indicate velocity of discharge in fps.

g. Solution.

(1) $Q_t = 28.0$ cfs.

(2) From table IX, $V_{max} = 5\text{–}7$ fps.

(3) Cross section, figure 46, $S = 1.5$ percent.

(4) S/S (embankment slope) $= 10:1$.

(5) Depth of fill:

$f_1 = 598.6' - 594.0' = 4.6'$

$a_1 = 10 \times 4.6' = 46.0'$

$f_2 = 46.0' \times 1.5\% = 0.7'$

$f_{tot} = 4.6' + 0.7' = 5.3'$

(6) Culvert length:

$b = 46.0' + 50.0' + 46.0' = 142.0'$

$y = 142.0' \times 1.5\% = 2.13'$

$$x = \frac{y}{S/S\ \text{ft/ft-}S_{cul}\ \text{ft/ft}}$$

$$= \frac{2.13'}{0.1\text{–}0.015} = 25.1'$$

$L = 142.0' + 25.1' + 2.0' = 169.1'$ or 170 feet, since pipe is ordered in even foot lengths.

Culvert design	Tot fill	Pipe diam	Cover req'd	S %	Q_p cfs	Q_t cfs	Pipes req'd	Outlet V fps	ft pipe req'd
1	5.3′	12″	1.0′	1.5	2.3	28.0	13	4	2210
2	5.3′	18″	1.2′	1.5	6.7	28.0	5	5	850
3	5.3′	24″	1.5′	1.5	14.0	28.0	2	6	340
4	5.3′	30″	2.0′	1.5	25.0	28.0	2	7	340
5	5.3′	36″	2.5′	1.5	Eliminated, in sufficient cover.				

Use: 2–24″ CMP pipes. This is the most economical design without considering a drop inlet.

h. Design of Pipe Culvert With a Submerged Inlet.

(1) Determine the rate of runoff that the culvert must drain, or, in the case of ponding, the drain inlet capacity, Q_d.

(2) Determine length of culvert as in *e*(7) above.

(3) Determine the head on the culvert (fig. 45).

(4) Determine from figure 48 the size of pipe, pipes, or box culvert required to handle the runoff, or the Q_d, if ponding is involved.

(5) Compute the discharge velocity (V) in feet per second by the equation $V = Q/A$. If the discharge velocity is greater than the maximum permissible velocity for the outfall (table IX), or the height of the water, in feet, above the top of the culvert inlet is less than 0.222 V^2 for CMP or less than 0.017 V^2 for concrete pipe or boxes,

either select pipes of larger diameter or decrease the slope of the culvert.

i. Example Problem, Submerged Inlet, Outlet Unsubmerged.

(1) Required: the size and number of pipes required to handle 210 cfs, through culvert at station 42 + 40 (fig. 47).

(2) The outfall from culvert is a natural drainage channel with dense turf in a GP soil; V_{max} (table IX) is 6–8 fps.

(3) 42″ and 48″ cast-concrete pipes are available.

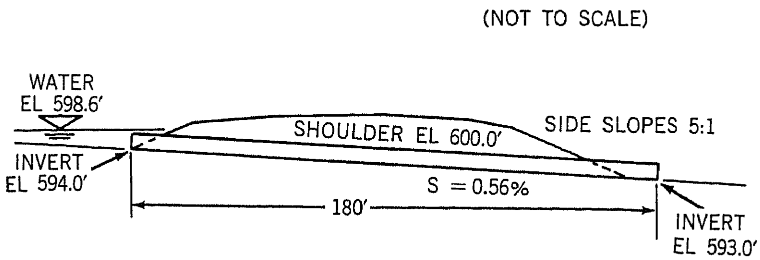


Figure 47. Cross section at station 42 + 40.

j. Solution.

(1) Head (H), see note, figure 48.

$$H_{42''} = 598.6' - (593.0' + 3.5') = 2.1'$$
$$H_{48''} = 598.6' - (593.0' + 4.0') = 1.6'$$

(2) Capacity of available pipes (from fig. 48):

(*a*) On cast concrete pipe portion of nomograph draw a horizontal line from the intersection of the 180′L line and the 42″ pipe line (point A) to the turning point line (point B). From B draw a line to an "H" of 2.1′ (point C). The intersection of this line with the "Q" portion of the nomograph shows the maximum discharge of one 42″ pipe to be about 78 cfs. For 210 cfs three would be required.

(*b*) Similarly for the 48″ pipe, 180′ length and 1.6′ H we proceed from D to E to F and find that one 48″ pipe would have a capacity discharge of about 92 cfs. Again three would be required for 210 cfs.

(3) Check smaller pipe for possible excessive outfall velocity.

(*a*) Since three pipes are used assume each will carry one-third the total Q or 70 cfs.

(*b*) $$V = \frac{Q}{A} = \frac{70}{\frac{\pi}{4}(3.5)^2} = \frac{70}{\frac{3.1416(12.25)}{4}} = 7.27 \text{ fps}$$

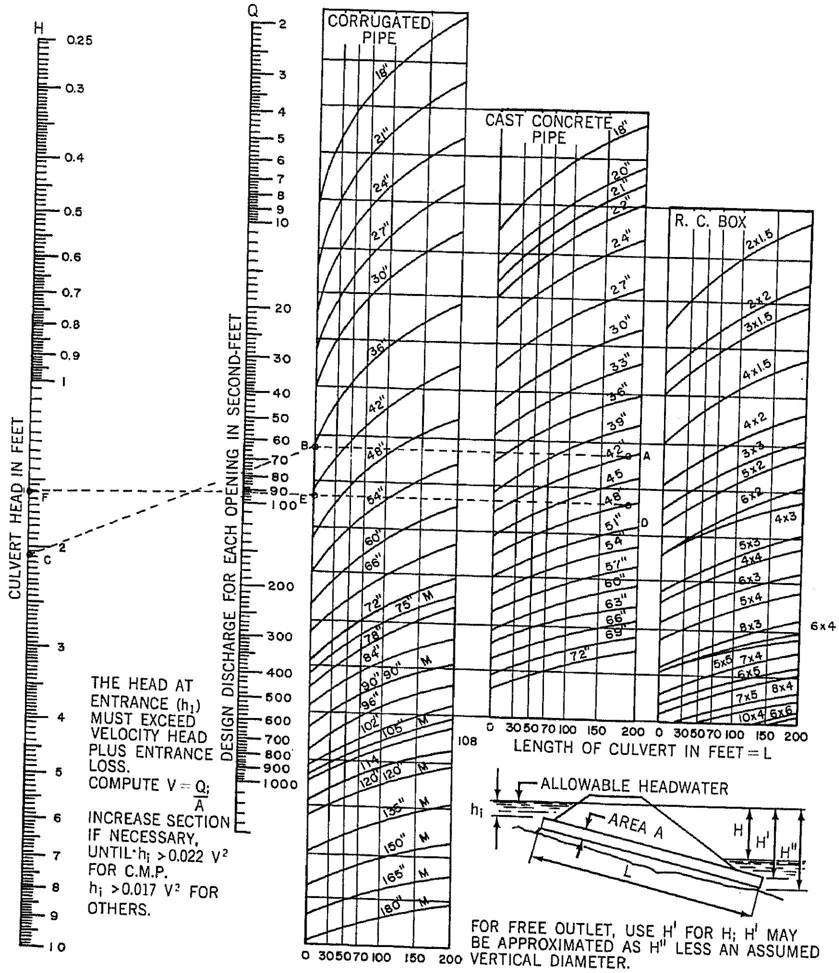


Figure 48. Nomograph for diameter or dimensions of a culvert flowing full with ponding at the inlet.

(4) Check to see that $h_i > 0.017\ V^2$ (see note, fig. 48)
$h_i = 598.6' - (594.0' + 3.5') = 1.1'$
$0.017 \times 7.27^2 = 0.89$

(5) Since allowable outfall velocity is not exceeded, and $h_i > 0.017\ V^2$, use three 42″ pipes as the most economical available size.

22. Excavation of Ditches With Equipment

Open ditches with sloping sides may be excavated with a grader (fig. 49), dozer, scraper, power shovel, backhoe, or dragline, depending on the size of the ditch and the prevailing working con-

ditions. In areas where the soil is practically free of stone, boulders, or hard stratified material, narrow trenches or ditches with vertical sides normally are excavated with the type of ditcher shown in figure 50. This machine makes better progress if it travels downgrade while digging, but if water will accumulate in the trench during construction the trench must be excavated uphill to avoid working in water. Characteristics and use of equipment mentioned in this paragraph are covered in TM 5–252.

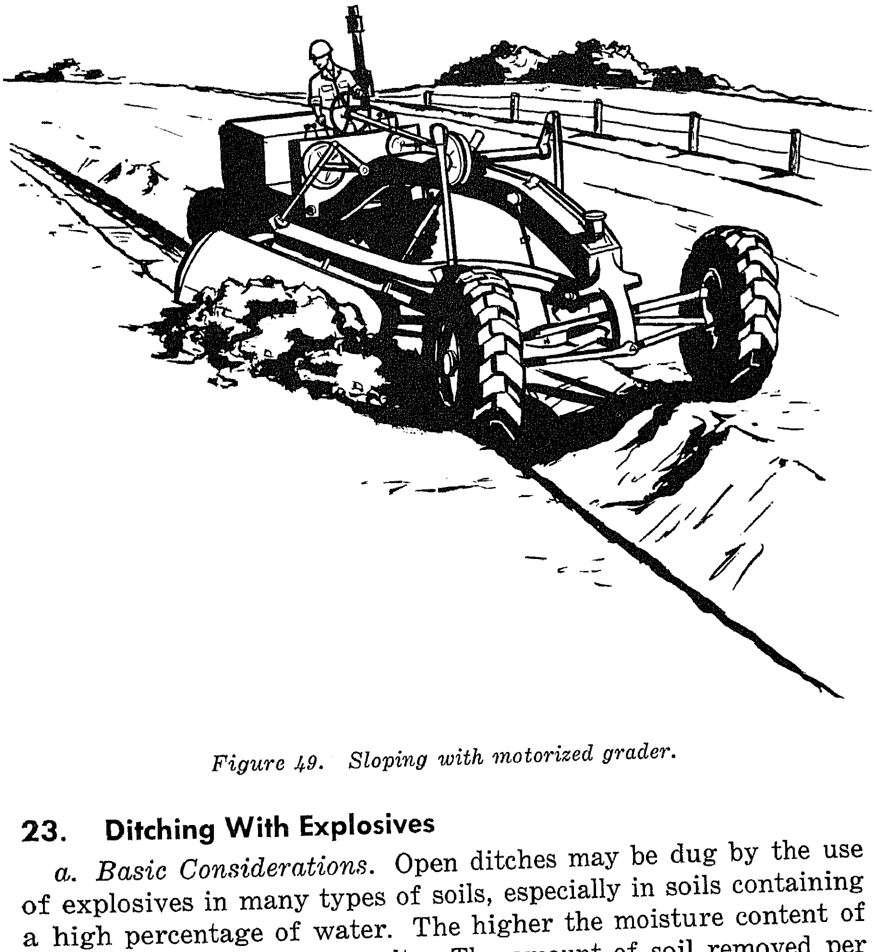


Figure 49. Sloping with motorized grader.

23. Ditching With Explosives

a. Basic Considerations. Open ditches may be dug by the use of explosives in many types of soils, especially in soils containing a high percentage of water. The higher the moisture content of the soil, the better the results. The amount of soil removed per pound of explosive ranges from 1/3 cubic yard to 1⅔ cubic yards, depending upon the method of blasting, the quantity of explosives used, and the type of material being blasted. The side slopes will be approximately 45°. Ditching with explosives has the following advantages: ability to dig successfully where conditions are too

Figure 50. Ditcher digging trench.

difficult for other methods, rapid completion of the job, minimum of equipment required, adaptability to both large and small ditches, and simplicity. Blasting is not effective in loose, dry sand or gravel.

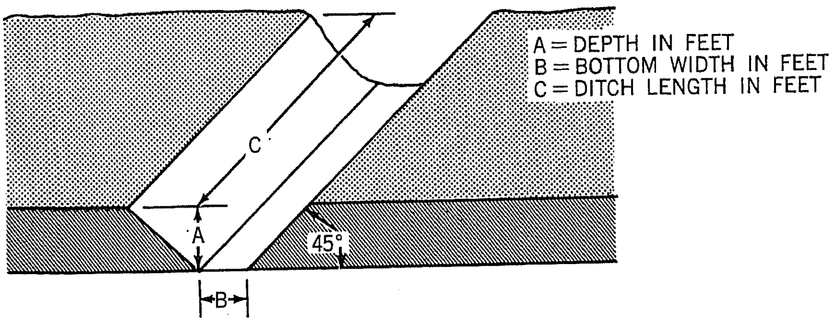
b. Quantity of Explosives. For quick estimates, the amount of explosives required to blast a ditch of known dimensions is roughly 1 pound of explosive for each cubic yard of material moved. The usual cross section is a trapezoid. To calculate the volume in cubic yards for the length of a ditch the following formula is used:

Volume (cu yd) for length of ditch

$$= \left(\frac{(B + 2A) + B}{2}\right) \times \left(\frac{A \times C}{27}\right)$$

$$= (A + B) \times \left(\frac{A \times C}{27}\right)$$

A, B, and *C* are as defined in figure 51 and are expressed in feet. Since the volume in cubic yards is roughly equal to the number of pounds of explosives required, the quantity of explosives is found directly from the formula.



$$\text{VOL. CU. YDS. FOR LENGTH DITCH} = \frac{(B + 2A) + B}{2} \times \frac{A \times C}{27}$$

Figure 51. Shape and slope of a blasted ditch.

c. Loading Methods and Patterns. Loading methods and patterns are shown in figures 52 through 55.

d. Propagation Method.

(1) *Basic features.* The propagation method is very simple, but it can only be used in wet soils and with straight or special ditching dynamite. It can be employed in swampy areas, where the surface is covered with heavy stumps and 1 to 2 feet of water. If the underlying soil

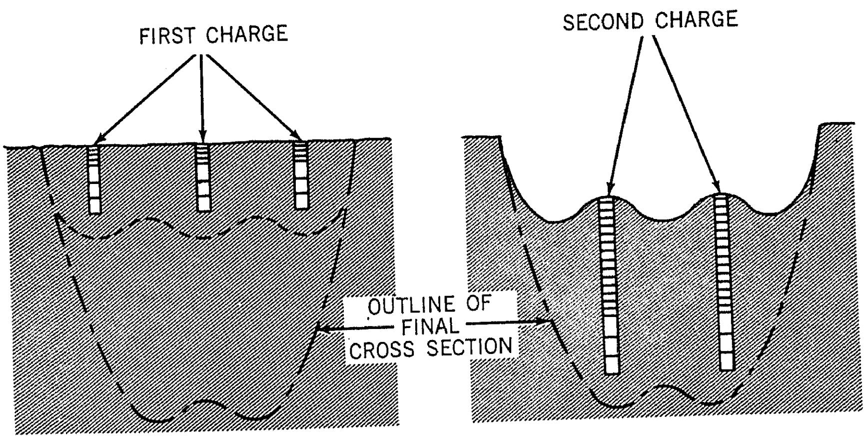


Figure 52. Method of loading a deep, narrow ditch.

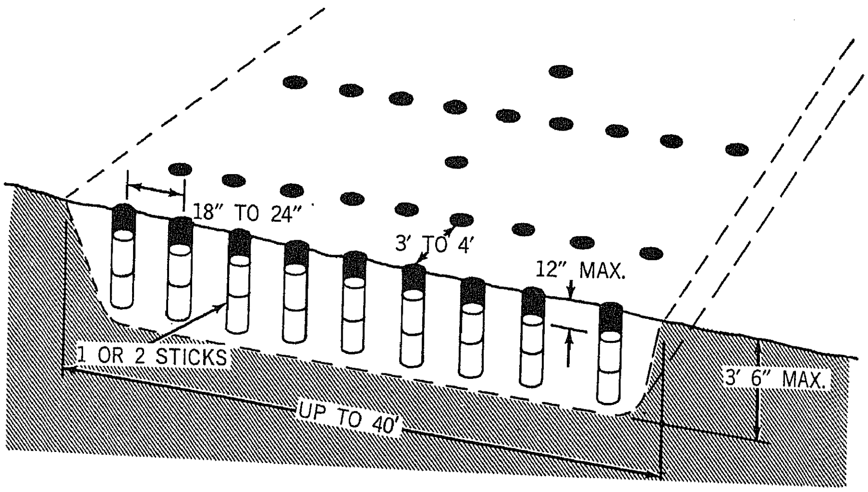


Figure 53. The cross-section method of loading to clean and widen ditches.

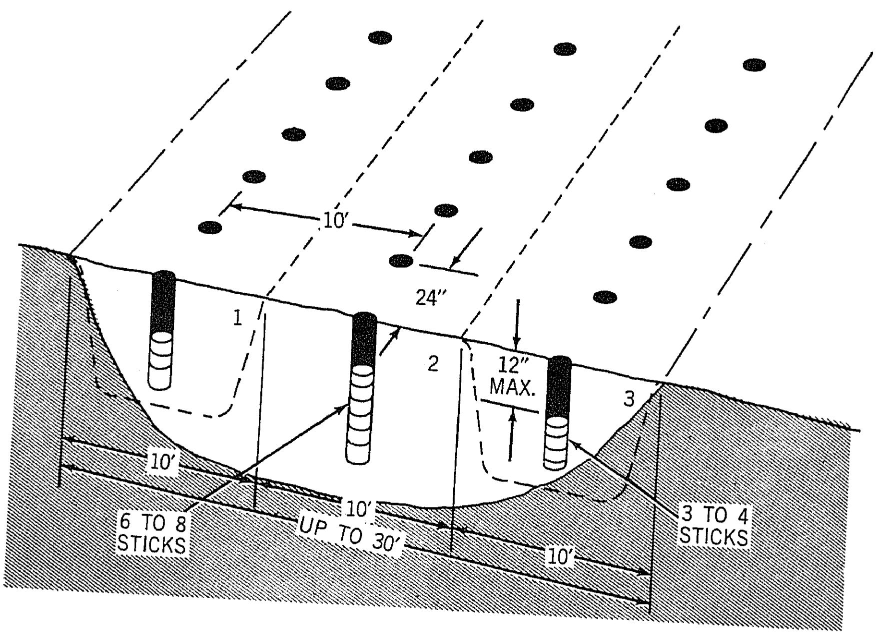


Figure 54. Relief method of loading for shallow ditches. Ditches 1 and 3 are blasted first to relieve the charge in ditch 2.

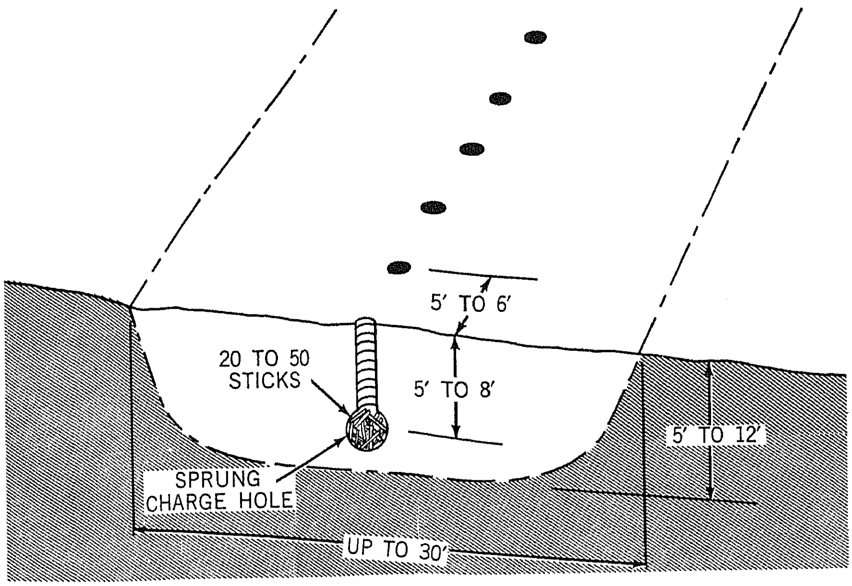


Figure 55. Post-hole method of loading for shallow ditches in mud.

is firm, it often works under even greater depths of water. In this method, only the first hole is primed, unless there is an obstruction such as a stump or rock separating the charges. The concussion of the explosion of this one primed charge propagates the detonation through the wet earth and sets off the entire line of charges. The priming may be done with either an electric blasting cap or a blasting cap and a time fuze.

(2) *Test shots.* Make a few trial shots to ascertain the best depth and spacing for holes. Generally, for ditches 3 to 3½ feet deep, charges should be placed 2 to 2½ feet deep should be placed 1 to 2 feet above the desired bottom of the ditch. In no case are the charges placed more than 24 ditch. In no case are the charges placed more than 24 inches apart. Begin with holes 2 feet deep and 18 inches apart. One extra charge should be placed in the primer hole and in each hole adjoining the primer. Correct loading will lift the soil at least 200 feet, leaving a clean ditch.

(3) *Amount of charge and size of ditch.* In soil with few roots, small ditches about 2 feet deep and 4 feet wide at the top are dug with ½-pound charges spaced 18 inches apart. Larger ditches are dug with 2- or 4-pound charges in each hole. A second and third line of charges parallel

to and 4 to 5 feet from the first line are used to make wide ditches. Each line of charges must be primed and it is advisable to overload the first or primed charge 1 to 2 pounds.

(4) *Charge holes.* The holes are bored along the ditch centerline. The charge line can be run with a transit or chalk line. When using a transit, the grade of the trench can be accurately controlled by checking the hole depth every 5 to 10 holes and at each change in grade. Holes are put down using a sharp punch or a quicksand punch. Holes are loaded and tamped at once to prevent cave-ins and to insure that the charge is at the correct depth.

e. Electric Method. The electric method permits the blasting of ditches in soil that is too dry or too heavy for the successful use of the propagation method. It can be employed in almost any type of soil except dry sand. An electric blasting cap is inserted in each charge. These caps are connected in a series or parallel-series circuit and detonated by means of a blasting machine. The rated capacity of the machine determines the number of charges which can be detonated at one time, and this in turn controls the length of the ditch. Loading and wiring plans are explained and illustrated in figure 56. The amount of charge for various size ditches is given in *d*(3) above. As an alternative to priming each charge with a cap, all charges may be primed with detonating cord and fired instantly using one cap.

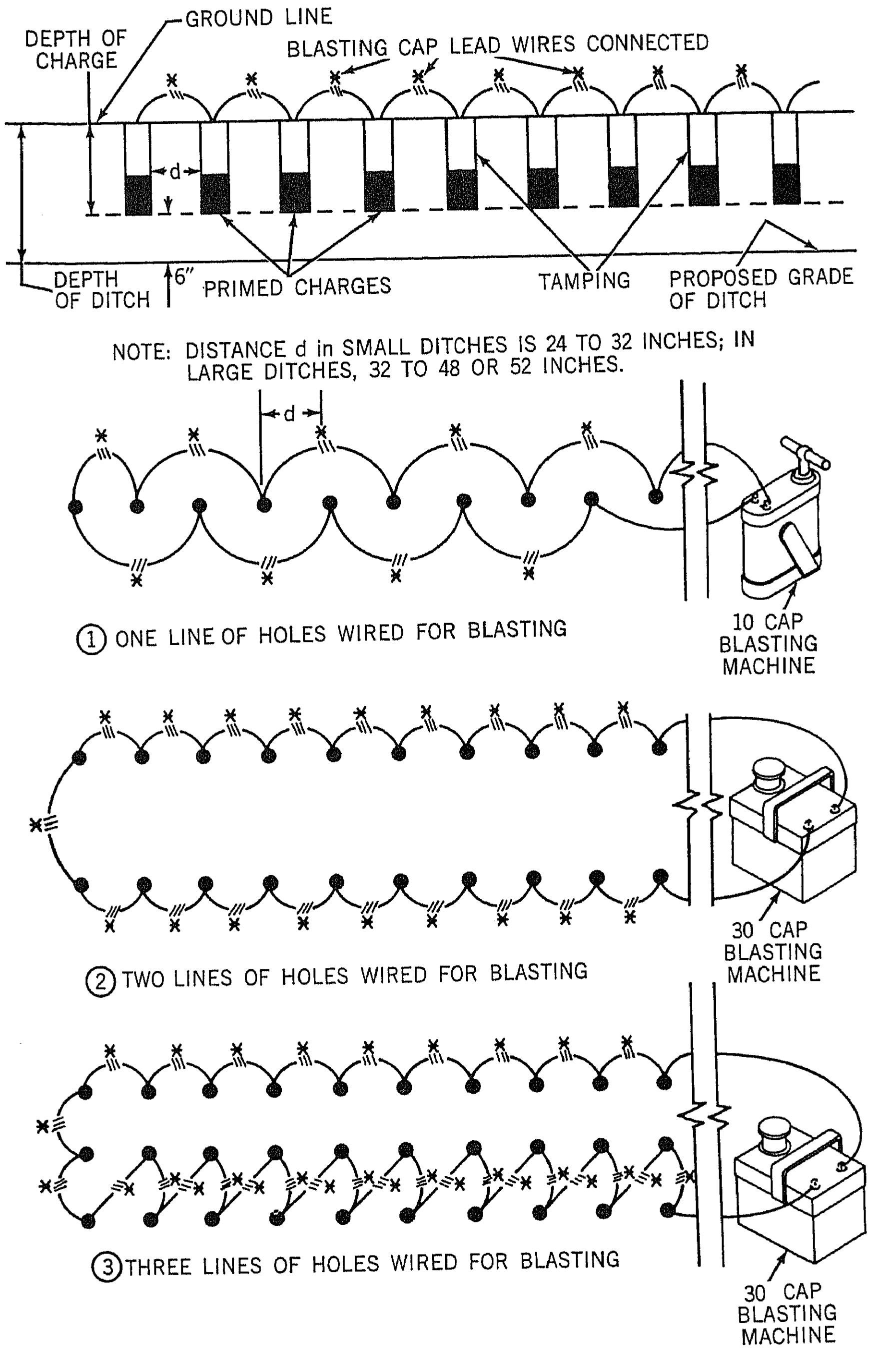


Figure 56. Loading and wiring plans for one, two, and three lines of charges.

CHAPTER 5

SUBGRADES AND BASE COURSES

24. Sources of Value in Planning Soil Surveys

a. Many sources of information concerning soils may be available for the area in which an engineering construction project is to be carried out. These sources include engineer intelligence reports, geologic and topographic maps and reports, agricultural soil maps and reports, and aerial photographs.

b. These sources of information may be the only ones available for objective areas held by the enemy and are of great value in strategic and tactical planning. From the standpoint of engineering construction, this information is of importance in two general ways. First, a study of this information, when available, will aid in securing a broad understanding of soil conditions and associated engineering problems which may be encountered in the area. Second, such information is of great value in planning, conducting, and interpreting the results of detailed soil-exploration programs which are necessary for design and construction. Properly used, available reports and maps are of value in holding soil-sampling operations to a minimum.

25. Purpose of Soil Surveys

Detailed soil surveys are conducted in order to determine the location, nature, and condition of soil layers at the site; to obtain samples for examination, classification, and testing; to develop the soil profile (par. 40); to determine field drainage characteristics, including the location of the water table; and to determine the location of rock layers. For the purpose of this discussion, three types of soil surveys are considered. Two of these are the hasty survey, which is made when time is very limited or under expedient conditions, and the deliberate survey, which is made when more time is available. Most of the discussion is devoted to the deliberate survey, and this may be regarded as the standard type of soil survey for construction purposes. The third type of survey is concerned with the exploration of sources of borrow materials.

26. Hasty Soil Survey

a. The hasty soil survey is made when limitations of time and the military situation do not permit a more detailed examination of soil conditions. It should be preceded by a careful study of the sources of information described in paragraph 24, if circumstances permit.

b. A trained person may make limited observations of soil conditions at the planned site of construction from the air. Significant observations may be noted mentally, in written notes, or upon available maps and photographs. Careful air observation gives an overall picture which is very difficult to secure from the ground, since important features which often cannot be detected from the ground may be seen easily from the air. This is particularly true in rough or wooded country.

c. Rapid ground observation along the proposed road location or at the site of a proposed airfield may also yield valuable information when circumstances do not permit a deliberate soil survey. The soil profile may be observed along the natural banks of streams, eroded areas, bomb craters, and other exposed places. Loose surface soil should be scraped off before the examination is made. Field identification (pars. 31–38) may be made of the observed soils. Samples may be taken from exposed soils for testing in a field laboratory, although sampling and testing are normally held to a minimum in this type of soil survey. Surface soils may be exposed by the use of a shovel or pick, particularly in areas of questionable soils or at critical points in the location. Soils identified in the hasty survey may be located by field sketches or on available maps or photographs. TM 5–530 covers the nature and use of the soil reconnaissance kits normally utilized in this type of soil survey.

d. Soil properties may be inferred from field identification and the values given in figure 61. As construction is carried out, additional soil studies must be made to amplify the information gained in the hasty survey. Changes can then be made in location, design, and construction as soil conditions dictate and as circumstances permit.

27. Deliberate Soil Survey

A deliberate soil survey is made when time permits, in order to gain the detailed information needed for location, design, and construction. Time may be a factor in these surveys, also, and their scope may have to be limited in certain cases. A deliberate survey is required for major construction projects. It is frequently performed at about the same time that detailed topographical information is being obtained, so that results of the soil survey may be integrated with other pertinent information. Sampling and testing which may be necessary to control the conduct of construction operations, such as compaction, are not considered a part of the soil survey.

28. Methods of Exploration

The principal method of exploration used in soil surveys for

roads, airfields, and borrow areas is the use of the hand auger. Other methods are the use of test pits and various samplers to relatively shallow depths. The use of this equipment is described in detail in TM 5–530. Other methods of exploration are described briefly in TM 5–541.

a. In road, airfield, and borrow area investigations, auger holes or test pits are dug at strategic points to obtain the maximum possible information with the least number of borings or pits. The strategic points are soil boundaries, grade changes, and other changes in soil conditions as determined by a study of air photos. Arbitrary spacing of these borings or pits at regular intervals does not give a true picture of the subgrade soil and does not locate the soil changes. A staggered arbitrary spacing of the holes based on information derived from air photographs is recommended. This procedure, in addition to providing a longitudinal profile, will provide a cross-sectional view of soil conditions beneath the road, runway, or borrow area. Additional holes may often be needed to further supplement the soil profile. In road locations, auger holes or test pits should be dug along the centerline in areas where large cuts or fills are to be made.

b. In planning the layout of test holes, particular attention must be given to critical points, that is, high and low points on the existing ground profile, and to points at which breaks occur in the surface profile, since such changes are frequently indicative of changes in subsurface soils.

c. In cut areas, all holes should extend at least 4 feet below final subgrade elevation. In fill areas, they should extend 4 feet below the natural ground elevation. As many holes as necessary should be extended to a reasonable depth in an effort to locate the groundwater table, if practicable. Depths of holes may well be increased in situations in which heavy loads are expected. Such a situation may occur, for example, when a high fill is to be built over a soft natural soil. Reasonably exact knowledge of the subsurface soil may make it possible to avoid excessive consolidation or failure in shear.

d. It should be remembered that it is always better to dig too many holes than too few. The assumption of uniform conditions where they do not exist may have disastrous consequences, and is not a risk worth the saving of a few test holes.

29. Locating, Numbering, and Recording Samples

The engineer in charge of the soil survey is responsible for properly surveying, numbering, and recording each auger boring, test pit, or other exploratory sampling. A log is kept of each test hole which shows the elevation (or depth below the surface) of

the top and bottom of each soil layer, the field identification of each soil encountered, and the number and type of each sample taken. Other information which may be included in the log is that relating to the density of each soil as compared to the maximum modified AASHO density, changes in moisture content, depth to ground water, and depth to rock.

30. Soil Samples

Bag, composite, moisture content, and undisturbed samples may be taken during a soil survey for a road, an airfield, or a borrow area as required. Details of the methods used in obtaining and preserving samples are contained in TM 5–530.

31. Physical Characteristics of Soils

a. Particles. Soils are made up of particles or grains that range in size from fairly large rock masses to microscopic particles. Particle size and particle-size distribution are important index properties, particularly for the coarser soils. Other properties such as plasticity (par. 32) are usually of greater importance in the case of fine-grained soils.

b. Particle Sizes. Particles are sorted into sizes by the use of sieves, which are merely screens attached to shallow circular containers. Sieves which are commonly used by the military engineer have square openings and are designated as 1-, ¾-, and ¼-inch sieves and U.S. Standard No. 4, 10, 20, 40, 60, 100, and 200 sieves. The size of openings which correspond to certain of the above sieve sizes is shown on the plot of figure 57. The larger sieve sizes, those with openings of ¼-inch or larger, are designated by the distance between the wires which form the openings. The smaller sieves are designated by the number of openings per linear inch. For example, the clear distance between wires in a ¼-inch screen is 6.35 mm. A No. 4 sieve has four openings per inch, and the clear distance between the wires is 4.15 mm.

c. Soil Groups. Soils may be divided into several different groups according to the range of soil-particle sizes included in each group. The grouping used in the Unified Soil Classification System, described in paragraph 33, is shown in table XII. Coarse gravel particles are comparable in size to a small orange, a lemon, an egg, or a walnut, whereas fine gravel ranges from walnut down to about pea size. Sand particles range in size from that of rock salt, through table salt or granulated sugar to powdered sugar. Particles passing a No. 200 sieve (fines) are designated as silt or clay, depending on their plasticity characteristics (par. 32).

d. Effect of Particle Size on Soil Density. Certainly one of the factors that affects the bearing capacity of soil is the density

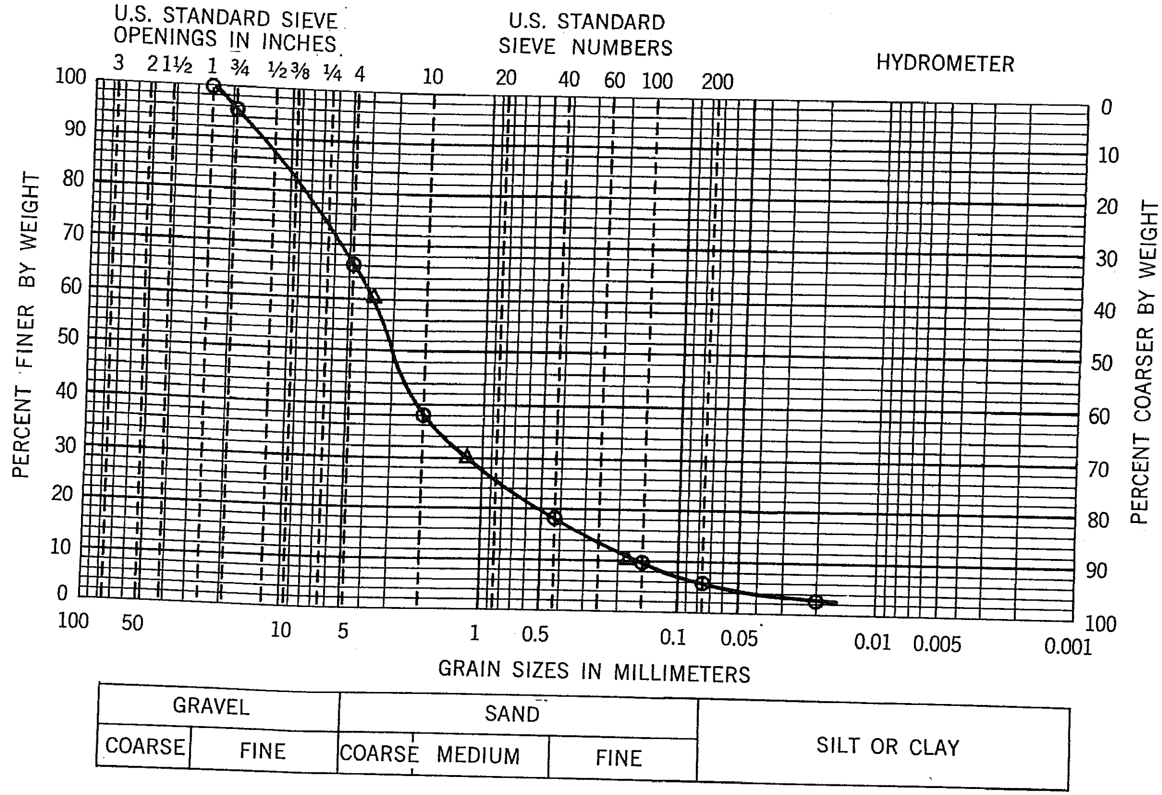


Figure 57. Grain-size distribution curve.

(weight per cubic foot) of the soil, especially when compacted. It has been found empirically that, generally speaking, coarse-grained soils can be compacted to greater densities than fine-grained soils. This is to be expected from the fact that if it were possible to remove some of the large particles without disturbing the surrounding material, and replace them with as large an amount of smaller particles as possible, a certain amount of void spaces would be introduced where solid substance had previously existed. This is why soils with a fair share of larger particles are usually denser; the volumes occupied by the large particles have, for practical purposes, no voids.

Table XII. Definition of Soil Groups on the Basis of Grain Size (Unified Soil Classition System)

	U.S. standard sieve size	
	Passing—	Retained on—
Cobbles	(*)	3-inch
Gravel	3-inch	No. 4
Sand	No. 4	No. 200
Fines	No. 200	—
Organic material	(**)	(**)

*Usually 40 inches maximum size due to capability of crushing equipment.
**No size boundary.

e. Particle Shape. The shape (or shapes) of the particles in a soil mass has an important influence on its strength and stability Two general shapes are normally recognized:

(1) *Bulky.* Gravel, sand, and silt particles, although differing greatly in size, are all bulky as to shape. The term includes all particles which are relatively similar in all three dimensions as contrasted with flaky grains in which one of the dimensions (thickness) is very small as compared with the other two. A book is an example of a bulky-shaped object since the length is not more than several times the thickness. A page of a book, on the other hand, illustrates a flaky-shaped object, where one of the dimensions is very much smaller than the other two. Four subdivisions of the bulky shape, in the order of desirability for construction, are:

(*a*) Angular particles are those which have been recently broken up and are characterized by jagged projections, sharp ridges, and flat surfaces. Angular particles of gravel and sand are generally the best materials for construction because they interlock well. Such particles are seldom found in nature. Angular materials

may be produced artificially by crushing, but because of the time and equipment required for crushing operations, natural materials with more rounded grain shapes are frequently used.

(*b*) Subangular particles are those that have been weathered to the extent that the sharper points and ridges have been worn off. The particles are still very irregular in shape with some flat surfaces.

(*c*) Subrounded particles result when weathering has progressed to a further degree. They are still somewhat irregular in shape but have no sharp corners and very few flat areas. Materials with this shape are frequently found in streambeds. They may be composed of hard, durable particles which are adequate for most construction needs.

(*d*) Rounded particles are those from which all projections have been removed and few irregularities in shape remain. The particles approach spheres of various sizes. Rounded particles are usually found in streambeds or beaches where repeated wave action has tended to produce, in many cases, almost perfectly rounded particles which are usually uniform in size.

(2) *Flaky.* Flaky particles are flat, platelike structures which are very thin relative to their breadth and length. It should be apparent that in a flaky particle the relationship between its surface or area and its weight is quite different from that in the case of a bulky particle. The greatly increased surface area provides a much greater contact area for moisture and is largely responsible for many of the characteristics which are associated with the flaky particles of clay soils.

f. Types of Soil Gradation. Since few natural soils consist of grains of uniform size, it is desirable to evaluate the percentages by weight of different sizes represented in a soil sample, especially in regard to coarse-grained soils.

(1) *Effective size and coefficient of uniformity.* The grain size which corresponds to 10 percent on a grain-size distribution curve of the type in figure 57 is called Hazen's effective size and is designated by the symbol D_{10}. For the soil shown, D_{10} is 0.17-mm. The coefficient of uniformity is defined as the ratio between the grain diameter corresponding to 60 percent on the curve D_{60} and D_{10}. It is given by the equation $C_u = \frac{D_{60}}{D_{10}}$. For the soil shown, $D_{60} = 4.0$-mm, and $C_u = 4.0/0.17 = 23.5$.

The uniformity coefficient is used in judging gradation, as is indicated in the following discussion.

(2) *Coefficient of curvature.* Another quantity which may be used to judge the gradation of a soil is the coefficient of curvature C_c. It is given by the expression:

$$C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$$

in which D_{60} and D_{10} have meanings previously given, and D_{30} is the grain diameter corresponding to 30 percent on the grain-size distribution curve. For the soil of figure 57, D_{30} = 1.2 mm, and C_c = 2.1.

(3) *Well-graded soils.* The grain-size curves for four different soils are shown in figure 58. Two of these soils, the ones which are labeled GW and SW, are well graded. The symbols GW, SW, GP, and SP shown on the drawing refer to groups within the Unified Soil Classification System, which is described in paragraph 33. Well-graded soils are those which have a reasonably large spread between the largest and smallest particles, and have no marked deficiency in any one size. The grain-size curve of such a soil is marked by its smoothness and gradual changes in slope. To be well graded, under the Unified Soil Classification System, gravels must have a value of C_u greater than 4; sands, greater than 6. Both gravels and sands must have a value of C_c between 1 and 3 to be well graded. Gravels and sands which do not meet these criteria are termed poorly graded and are indicated by the symbols GP and SP.

(4) *Poorly-graded soils.* There are two general types of poorly-graded, coarse soils: uniform and skip-graded.

(*a*) *Uniform soils.* The curve which is marked SP in figure 58 is a poorly-graded, uniform soil. By uniform is meant that the particles are nearly all the same size, that is, the difference in size between the largest and smallest grains is small. The slope of the grain-size curve of such a soil is characteristically steep. Beach sands are typical of uniform soils.

(*b*) *Skip-graded soils.* The soil which is represented by the curve GP in figure 58 is also poorly graded. This material falls in the category of skip- or gap-graded soils. This means there is a deficiency in one size of particles. This gap in gradation produces a characteristic hump or plateau in the grain-size curve. The coefficient of uniformity may or may not disclose a skip-graded soil, but if it does not, the graphical rep-

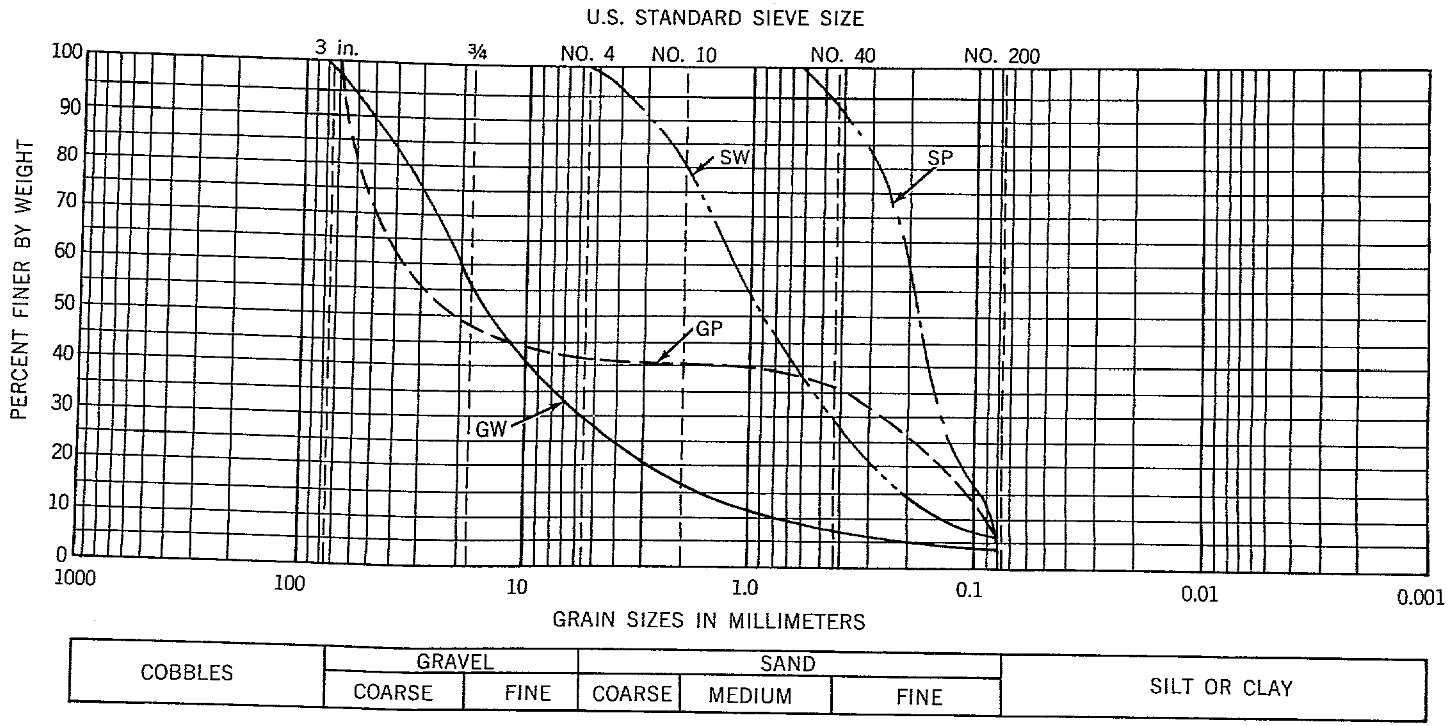


Figure 58. Typical well-graded and poorly graded soils.

resentation on the grain-size distributional curve will identify a skip- or gap-graded soil as poorly graded.

32. Soil Moisture and Related Soil Characteristics

The effects of particle size, shape, and gradation are not the only soil properties of concern to the soils engineer. On the contrary, an understanding of the way soils are affected by water is more important than any other single factor.

a. *Sources and Types of Soil Moisture.*

(1) *Surface water.* Surface water may enter from the sides of the road or airfield even though the facility has a relatively waterproof surface. To reduce this infiltration to a minimum, shoulders and ditches are usually designed to carry the surface water away at an acceptable rate. Poor maintenance, accumulated vegetation, and/or sedimentation may reduce the drainage efficiency to such an extent that water will frequently be ponded for a prolonged period of time in the drainage ditches. This water may then infiltrate the base and/or subgrade.

(2) *Subsurface water.*

(*a*) Water which percolates down from surface sources eventualy reaches a depth at which there exists some medium which restricts or stops the further percolation of the moisture. This medium is often bedrock, but it can be a layer of soil with such small void spaces that the amount of water that leaves the soil zone immediately above it is not as great as the quantity of water reaching that soil zone. In time the accumulating water will completely saturate the soil above the impervious or restricting medium and fill all the voids with water up to a depth below the ground surface that varies with the rate of evaporation and/or runoff capability. When this zone of saturation is under no pressure (except atmospheric) it is called free or gravitational water. The upper limit of the saturated zone is called the water table (par. 39). During a wet winter the ground water table will rise; conversely, it will fall during a hot, dry summer.

(*b*) Capillary action in a soil results in the "capillary fringe" immediately above the ground water table. The height of capillary rise depends upon several factors, but one of the most important is the pore structure of the soil. A fine-grained soil will develop a higher capillary fringe or zone than a coarse-grained

soil. In silts, capillary water sometimes rises as high as 10 feet and in clays as high as 30 feet.

(c) When wet soil is air dried in the laboratory, moisture is removed by evaporation until the hygroscopic moisture in the soil is in equilibrium with the moisture vapor in the air. The amount of moisture in air-dried soil, expressed as a percentage of the weight of the oven-dry soil, is called the hygroscopic-moisture content. Hygroscopic moisture films may be driven off from air-dried soil by heating the material in an oven at 100° to 110° C. (212° to 230° F.) until a constant weight is attained.

b. Moisture Content. This term is used in studying the effect of water upon soil. Moisture content is equal to the weight of water in the soil sample (wet weight minus oven-dry weight) divided by the dry weight. When multiplied by 100 the moisture content may be expressed as a percentage. The small letter w is usually used to represent this relationship:

$$w = \text{Moisture content in percent} = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

c. Plasticity and Cohesion. Two very important aspects of the engineering behavior of fine-grained soils are plasticity and cohesion. These phenomena are associated with the presence of adsorbed water films in the soil mass; adsorbed water is discussed in detail in TM 5–541; field tests are discussed in paragraphs 38 of this manual.

(1) *Plasticity.* By plasticity is meant the ability of a soil to deform under pressure without crumbling or breaking. Clay soils normally are plastic over a wide range of moisture contents. Coarse soils, such as clean sands and gravels, are nonplastic. Silts also are essentially nonplastic materials. All clays, on the other hand, are plastic and, since practically all fine-grained soils contain some clay, most of them are in some degree plastic. The degree of plasticity a soil possesses can be used as a general index to its clay content. Sometimes the terms "fat" and "lean" are used to describe the amount of plasticity. A lean clay is one that is only slightly plastic because it contains a large proportion of silt and/or fine sand.

(2) *Cohesion.* Soils which are highly plastic are also cohesive. That is, they possess some cohesion or resistance to deformation because of the surface tension present in the water films. Thus, wet clays can be molded into various shapes without breaking and will retain these

shapes, even though wet. Gravels, sands, and most silts are not cohesive; they are called *cohesionless soils.* Soils of this general class cannot be molded into permanent shapes and have little or no strength when dry and unconfined. Some of these soils may be slightly cohesive when damp. This is attributed to what is sometimes called apparent cohesion, which is also due to the surface tension in the water films between the grains.

d. Consistency Limits of Fine-Grained Soils. A fine-grained soil can exist in any one of several different states, depending upon the amount of water which is present in the soil. The boundaries between the different states in which a soil may exist are moisture contents which are called *consistency limits.* They are also called *Atterberg limits* after the Swedish soil scientist who first defined them. Thus, the *liquid limit* (LL) is the boundary between the liquid and plastic states. Above the liquid limit, the soil is presumed to behave as a liquid. The *plastic limit* (PL) is the boundary between the plastic and semisolid states. The numerical difference between the liquid limit and the plastic limit is called the *plasticity index* (PI). It is the range of moisture content over which soil is in a plastic condition. The *shrinkage limit* is the boundary between the semisolid and solid states. The Atterberg limits are important index properties of fine-grained soils. They are particularly important in classification and identification. They are also widely used in specifications to control the properties and behavior of soil mixtures.

e. Liquid Limit Defined. The liquid limit (LL) of a soil is its moisture content (w) at the boundary between the liquid and plastic states expressed to the nearest whole number. High values of the liquid limit indicate that the soil is highly compressible. TM 5–530 contains details of the test procedures to determine the liquid limit.

f. Plastic Limit. The plastic limit of a soil is its moisture (w) at the boundary between the plastic and semisolid states expressed to the nearest whole number. TM 5–530 contains details of the test procedures to determine the plastic limit.

g. Plasticity Index. The plasticity index of a soil is the numerical difference between the liquid and plastic limits. For example, if a soil has a liquid limit of 57 percent and a plastic limit of 21 percent, then the plasticity index, $PI = LL - PL$ or $57 - 21 = 36$. Sandy soils and silts have characteristically low values of the plasticity index, while most clays have higher values. Soils which have high values of this index are highly plastic and, in general, are highly compressible; they are also highly cohesive. Soils that have plastic limits equal to the liquid limit, or, as may occasionally

occur, have plastic limits slightly higher than the liquid limit, are reported as having a plasticity index of zero.

h. Shrinkage. Most plastic soils undergo a very considerable reduction in volume when their moisture content is reduced. The effect is most pronounced when the moisture content is reduced from complete saturation to very dry. This reduction in volume is called shrinkage and is greatest in clays. Some of these soils show a reduction in volume of 50 percent or more while passing from a saturated to an oven-dry condition. Cohesionless sands and gravels, on the other hand, show no change in volume with change in moisture content. The shrinkage of a clay mass may be attributed to the surface tension existing in the water films created during the drying process. When the soil is saturated, a free water surface exists on the outside of the soil mass and the effects of surface tension are not important. As the soil dries out because of evaporation, the surface water disappears and innumerable meniscuses are created in the voids which are adjacent to the surface of the soil mass. Tensile forces are created in each of these boundaries between water and air, which act upon the soil structure. In the case of the typically fairly dense structure of a sand or gravel, the forces are of little consequence and very little or no shrinkage results. In fine-grained soils, the soil structure is compressible and the mass shrinks. As drying continues, the mass attains a certain limiting volume. At this point the soil is still saturated. The moisture content at this stage is called the *shrinkage limit*. Further drying will not cause a reduction in volume, but may cause cracking as the meniscuses retreat into the voids. In clay soils the internal forces set up by drying may become very large. The existence of these forces also principally accounts for the rockline strength of a dried clay mass. Both silt and clay soils may be subject to detrimental shrinkage with disastrous results in some practical situations. For example, the uneven shrinkage of a clay soil may deprive a concrete pavement of the uniform support for which it is designed. Severe cracking or failure of the pavement may result when wheel loads are applied.

i. Swelling and Slaking. If water is again made available to a soil mass which has undergone shrinkage but is still saturated, it will enter the voids of the soil mass from the outside and reduce or destroy the internal forces previously described. Thus, a clay mass will absorb water and may expand or swell. The expansion force created as the water enters the soil may be very large. If the expansion is prevented, as by the weight of a concrete pavement, the expansion force may be sufficient to cause severe cracking of the pavement. If water is made available to the surface of

a mass of soil after the moisture content is below the shrinkage limit, the mass will generally simply disintegrate along the surface or slake. The phenomenon of slaking may be observed by putting a dry piece of clay into a glass of water. The mass will disintegrate along the surfaces and sometimes fall completely apart. Destruction problems associated with shrinkage and expansion to a detrimental degree are generally solved by one of the following methods:

(1) Taking steps to prevent excessive changes in moisture content,
(2) Stabilizing the soil, as discussed in paragraphs 51 through 70, or
(3) Removing the soils which are subject to these phenomena.

33. Unified Soil Classification System

In the Unified Soil Classification System all soils are divided into three major categories: coarse-grained soils, fine-grained soils, and highly organic soils. Coarse-grained soils are defined as those in which more than half the material, by weight, is larger than (retained on) a No. 200 sieve. Fine-grained soils are those in which half or more than half the material, by weight, is smaller than (passes) a No. 200 sieve. The third major category, highly organic soils, is not defined by numerical, mechanical-analysis criteria: these soils are identified by visual and manual inspection.

a. Coarse-Grained Soil Types. The coarse-grained soils are divided into two major divisions: gravels and sands. A coarse-grained soil is classed as a gravel if more than half the coarse fraction, by weight, (fraction retained on a No. 200 sieve) is retained on a No. 4 sieve. It is a sand if more than half the coarse fraction is smaller than a No. 4 sieve. The symbol G is used to denote a gravel, and the symbol S to denote a sand. In general practice there is no clear-cut boundary between gravelly and sandy soils; as far as behavior is concerned, the exact point of division is relatively unimportant. Where a mixture occurs, the primary name is the predominant fraction; the minor fraction is used as an adjective. For example, a sandy gravel would be a mixture containing more gravel than sand. For the purpose of systematizing the discussion, it is desirable to further divide coarse-grained soils into three groups on the basis of the amount of fines (materials passing a No. 200 sieve) which they contain.

b. Coarse-Grained Soils With Less Than 5 Percent Passing No. 200 Sieve. These soils may fall into the groups GW, GP, SW, or SP, as follows (refer to par. 31 for a discussion of the meaning of the terms well-graded and poorly-graded):

(1) *GW and SW groups.* In the GW groups are well-graded gravels and gravel-sand mixtures which contain little or no fines. The fines, if present, must not noticeably change the strength characteristics of the coarse-grained fraction, and must not interfere with its free-draining characteristics. The SW group contains well-graded sands and gravelly sands with little or no fines. The grain-size distribution curves marked GW and SW in figure 58 are typical of soils included in these groups. Definite laboratory classification criteria have been established to judge if the soil is well graded. For the GW group, the uniformity coefficient (C_u) must be greater than 4, and for the SW group the C_u must be greater than 6. For both groups, the coefficient of curvature (C_c) must be between 1 and 3 as shown in the upper right-hand portion of figure 59.

Figure 59. Unified Soil Classification System.
(Located in back of manual)

(2) *GP and SP groups.* The GP group includes poorly-graded gravels and gravel-sand mixtures containing little or no fines. In the SP group are contained poorly-graded sands and gravelly sands with little or no fines. These soils will not meet the gradation requirements established for the GW and SW groups and the fines, if present, must not interfere with free drainage properties. The grain-size-distribution curve marked GP in figure 58 is typical of a poorly-graded gravel-sand mixture, while the curve marked SP is a poorly-graded (uniform) sand.

c. Coarse-Grained Soils Containing More Than 12 Percent Passing No. 200 Sieve. These soils may fall into the groups designated GM, GC, SM, and SC. The use of the symbols M and C is based upon the plasticity characteristics of the material passing the No. 40 sieve. The liquid limit and plasticity index are used in specifying the laboratory criteria for these groups. Reference is also made to the plasticity chart shown in figure 59, which is based upon established relationships between the liquid limit and plasticity index for many different fine-grained soils. The symbol M is used to indicate that the material passing the No. 40 sieve is silty in character. M is taken from the Swedish Mo, which usually designates a fine-grained soil of little or no plasticity. The symbol C is used to indicate that the binder soil is clayey in character.

(1) *GM and SM groups.* Typical of the soils included in the GM group are silty gravels and poorly-graded gravel-sand-silt mixtures. Similarly, in the SM group are con-

tained silty sands and poorly-graded sand-silt mixtures. Gradation of these materials is not considered significant. For both these groups, the Atterberg limits must plot below the A-line of the plasticity chart in figure 59 or the plasticity index must be less than 4.

(2) *GC and SC groups.* The GC group includes clayey gravels and poorly-graded gravel-sand-clay mixtures. Similarly, SC includes clayey sands and poorly-graded sand-clay mixtures. Gradation of these materials is not considered significant. For both of these groups, the Atterberg limits must plot above the A-line (fig. 59) with a plasticity index of more than 7.

d. Borderline Coarse-Grained Soils. Coarse-grained soils which contain between 5 and 12 percent of material passing the No. 200 sieve are classified as borderline and given a dual symbol, such as GW-GM. Similarly, coarse-grained soils which contain more than 12 percent of material passing the No. 200 sieve, and for which the Atterberg limits plot in the shaded portion of the plasticity chart (fig. 59), are classified as borderline and require a dual symbol such as SM-SC. It is possible, in rare instances, for a soil to fall into more than one borderline zone and, if appropriate symbols were used for each possible classification, the result would be a multiple designation consisting of three or more symbols. This approach is unnecessarily complicated, and it is considered best to use only a double symbol in these cases, selecting the two that are believed to be most representative of the probable behavior of the soil. In cases of doubt, the symbols representing the poorer of the possible groupings should be used. For example, a well-graded sandy soil with 8 percent passing the No. 200 sieve, with LL of 28 and PI of 9, would be designated as SW-SC. If the Atterberg limits of this soil were such as to plot in the shaded portion of the plasticity chart (fig. 59) (LL 20 and PI 5), the soil would be designated either SW-SC or SW-SM, depending on the judgment of the engineer.

e. Fine-Grained Soils. The fine-grained soils are not classified on the basis of grain size but according to plasticity and compressibility. Laboratory classification criteria are based on the relationship between the liquid limit and plasticity index which is designated as the plasticity chart in figure 59. This chart was established by the determination of limits for many soils, together with an analysis of the relation between the limits values and the physical characteristics. Examination of the chart will show there are two major groupings of fine-grained soils. These are the L groups, which have liquid limits of less than 50, and the H groups, which have liquid limits in excess of 50. The symbols L and H

have general meanings of low and high compressibility, respectively. Fine-grained soils are further divided with relation to their position above or below the A-line of the plasticity chart.

(1) *ML and MH groups.* Typical soils of the ML and MH groups are inorganic silts; those of low liquid limit in the ML group, others in the MH. All of these soils plot below the A-line. In the ML group are included very fine sands, rock flours, and silty or clayey fine sands with slight plasticity. Loess-type soils usually fall into this group. Micaceous and diatomaceous soils generally fall into the MH group, but may extend into the ML group when their liquid limits are less than 50. The same statement is true of certain types of kaolin clays which have low plasticity.

(2) *CL and CH groups.* In these groups the symbol C stands for clay, with L and H denoting low or high liquid limits. These soils plot above the A-line and are principally inorganic clays. In the CL group are included gravelly clays, sandy clays, silty clays, and lean clays. In the CH group are inorganic clays of high plasticity, including fat clays, the gumbo clays of the southern United States, volcanic clays, and bentonite. The glacial clays of the northern United States cover a wide band in the CL and CH groups.

(3) *OL and OH groups.* The soils in these two groups are characterized by the presence of organic matter, hence the symbol O. All of these soils plot below the A-line. Organic silts and organic silt-clays of low plasticity fall into the OL group, while organic clays plot in the OH zone of the plasticity chart. Many of the organic silts, silt-clays, and clays deposited by the rivers along the lower reaches of the Atlantic seaboard have liquid limits between 40 and more than 100, and plot below the A-line. Peaty soils may have liquid limits of several hundred percent, but will plot well below the A-line.

(4) *Borderline soils.* Fine-grained soils with limits which plot in the shaded portion of the plasticity chart are borderline cases, and are given dual symbols, such as CL-ML. Several soil types exhibiting low plasticity plot in this general region on the chart, with no definite boundary existing between silty and clayey soils.

f. Highly Organic Soils, Pt Group. A special classification (Pt) is reserved for the highly organic soils, such as peat, which have so many undesirable characteristics from the standpoint of their behavior as foundations and their use as construction materials.

No laboratory criteria are established for these soils, since they generally can be readily identified in the field by their distinctive color and odor, spongy feel, and frequently fibrous texture. Particles of leaves, grass, branches, or other fibrous vegetable matter are common components of these soils.

g. Schematic Diagram. Other details of the Unified Soil Classification System are shown schematically in figure 60.

Figure 60. Schematic diagram of the Unified Soil Classification System.
(Located in back of manual)

34. Characteristics of Soil Groups

a. Use of Soil Groups in Construction Design. The properties desired in soils for subgrades and for base courses are: adequate strength, good compaction characteristics, adequate drainage, resistance to frost action in areas where frost is a factor, and acceptable compression and expansion characteristics. Certain of these properties, if inadequate in the soils available, may be supplied by proper construction methods. For instance, materials having good drainage characteristics are desirable, but if such materials are not available locally, adequate drainage may be obtained by installing a properly designed water-collecting system. Strength requirements for base-course materials to be used immediately under pavement of a flexible pavement structure are high, and only good quality materials are acceptable. However, low strengths in subgrade materials may be compensated for in many cases by increasing the thickness of overlying concrete pavement or of base materials in flexible pavement construction. From the foregoing brief discussion, it may be seen that the proper design of roads and airfield pavements requires the evaluation of soil properties in more detail than is possible by use of the general soils classification system. However, the grouping of soils in the classification system gives a general indication of their behavior in road and airfield construction. General characteristics of the soil groups pertinent to roads and airfields are presented in figure 61. Columns 1 through 3 show major soil divisions and all soil-group symbols; columns 4 and 5 show hatchings and colors to be used to designate each soil type in schematic representation of soil profiles; column 6 gives names of soil types; column 7 evaluates the performance (strength) of the soil groups when they are used as subgrade materials that will not be subject to frost action; column 8 makes a similar evaluation for the soils when they are used as base course materials; potential frost action is shown in column 9; compressibility and expansion characteristics are shown in column 10; column 11 presents drainage characteristics; column 12 shows types of compaction equipment which

perform satisfactorily on the various soil groups; column 13 shows ranges of unit dry weight for compacted soils; column 14 shows ranges of typical California Bearing Ratio (CBR) values to be anticipated for use in airfield design; and column 15 gives ranges of modulus of subgrade reaction (k). The various features presented are discussed in the following paragraphs.

Figure 61. Characteristics of soil groups.
(Located in back of manual)

b. Subdivision of Coarse-Grained Soil Groups. It will be noted in column 3, letter symbols, that the basic soil groups, GM and SM, have each been subdivided into two groups designated by the suffixes d and u. These suffixes represent desirable and undesirable base and subbase materials, respectively. This subdivision applies to roads and airfields only and is based on field observation and laboratory tests on the behavior of the soils in these groups. Basis for the subdivision is the liquid limit and plasticity index of the fraction of the soil passing the No. 40 sieve. The suffix d is used when the liquid limit is 25 or less and the plasticity index is 5 or less; the suffix u is used otherwise. Typical symbols for soils in these groups are GMd and SMu, etc.

c. Value of Soils as Subgrade or Base Course Materials. The descriptions in columns 7 and 8 give a general indication of the suitability of the soil groups for use as subgrade, subbase, or base course materials, provided they are not subject to frost action. In areas where frost heaving is a problem, the value of materials as subgrades will be reduced, depending on the potential frost action of the material, as shown in column 9. Proper design procedures should be used in situations where this is a problem. The coarse-grained soils in general make the best subgrade, subbase, and base materials. The GW group has excellent qualities as a subgrade and subbase, and is good as a base material. It is noted that the adjective "excellent" is not used for any of these soils for base courses; it is considered that the adjective "excellent" should be used in reference to a high quality processed crushed stone. Poorly-graded gravels and some silty gravels, groups GP and GMd, are usually only slightly less desirable as subgrade or subbase materials, and under favorable conditions may be used as base materials for certain conditions. However, poor gradation and other factors sometimes reduce the value of these soils to such extent that they offer only moderate strength and therefore their value as a base material is less. The GMu, GC, and SW groups are reasonably good as subgrade or select materials, but are generally poor to not suitable as bases. The SP and SMd soils usually are considered fair to good subgrade and subbase materials but in general are poor to not suitable for base

materials. The fine-grained soils range from fair to very poor subgrade materials as follows: silts and lean clays (ML and CL), fair to poor; organic silts, lean organic clays and micaceous or diatomaceous soils (OL and MH), poor; fat clays and fat organic clays (CH and OH), poor to very poor. These shortcomings are compensated for in flexible-pavement design by increasing the thickness of overlying base material, and in rigid-pavement design by increasing the pavement thickness or by adding a base-course layer. None of the fine-grained soils are suitable as subbases under bituminous pavements, but soils in the ML and CL groups may be used as select materials. The fibrous organic soils (group Pt) are very poor subgrade materials and should be removed wherever possible; they are not suitable as subbase and base materials.

d. Potential Frost Action. The relative effects of frost action on the various soil groups are shown in column 9. Regardless of the frost susceptibility of the various soil groups, two conditions must be present simultaneously before frost action will be a major consideration. These are a source of water during the freezing period and a sufficient period for the freezing temperature to penetrate the ground. Water necessary for the formation of ice lenses may become available from high ground-water table, capillary supply, or water held within the soil voids, or through infiltration. The degree of ice formation that will occur in any given case is markedly influenced by environmental factors such as topographic position, stratification of the parent soil, transitions into cut section, lateral flow of water from side cuts, localized pockets of perched ground water, and drainage conditions. In general, the silts and fine silty sands are the worst offenders as far as frost is concerned. Coarse-grained materials with little or no fines are affected only slightly if at all. Clays (CL and OH) are subject to frost action, but the loss of strength of such materials may not be as great as for silty soils. Inorganic soils containing less than 3 percent by weight of grains finer than 0.02 millimeter in diameter are generally nonfrost-susceptible. Where frost-susceptible soils are encountered in subgrades and frost is a definite problem, three acceptable methods of design of pavements are available. One method is to place a sufficient depth of acceptable granular material over the soils to prevent or limit freezing in the subgrade and thereby prevent the detrimental effects of frost action, or reduce it to acceptable amounts. A second method is to use a reduced depth of granular material, thereby allowing freezing in the subgrade, and to base the design on the reduced strength of the subgrade during the frost-melting period. Another method that may be used in many cases, is to provide appropriate drain-

g. Solution.

(1) $Q_t = 28.0$ cfs.

(2) From table IX, $V_{max} = 5\text{–}7$ fps.

(3) Cross section, figure 46, $S = 1.5$ percent.

(4) S/S (embankment slope) $= 10:1$.

(5) Depth of fill:

$$f_1 = 598.6' - 594.0' = 4.6'$$
$$a_1 = 10 \times 4.6' = 46.0'$$
$$f_2 = 46.0' \times 1.5\% = 0.7'$$
$$f_{tot} = 4.6' + 0.7' = 5.3'$$

(6) Culvert length:

$$b = 46.0' + 50.0' + 46.0' = 142.0'$$
$$y = 142.0' \times 1.5\% = 2.13'$$
$$x = \frac{y}{S/S\ \text{ft/ft-}S_{cul}\ \text{ft/ft}}$$
$$= \frac{2.13'}{0.1\text{–}0.015} = 25.1'$$

$L = 142.0' + 25.1' + 2.0' = 169.1'$ or 170 feet, since pipe is ordered in even foot lengths.

Culvert design	Tot fill	Pipe diam	Cover req'd	S %	Q_p cfs	Q_t cfs	Pipes req'd	Outlet V fps	ft pipe req'd
[illegible]	5.3′	12″	1.0′	1.5	2.3	28.0	13	4	2210
[illegible]	5.3′	18″	1.2′	1.5	6.7	28.0	5	5	850
[illegible]	5.3′	24″	1.5′	1.5	14.0	28.0	2	6	340
4	5.3′	30″	2.0′	1.5	25.0	28.0	2	7	340
5	5.3′	36″	2.5′	1.5	Eliminated, in sufficient cover.				

Use: 2–24″ CMP pipes. This is the most economical design thout considering a drop inlet.

h. Design of Pipe Culvert With a Submerged Inlet.

(1) Determine the rate of runoff that the culvert must drain, or, in the case of ponding, the drain inlet capacity, Q_d.

(2) Determine length of culvert as in *e*(7) above.

(3) Determine the head on the culvert (fig. 45).

(4) Determine from figure 48 the size of pipe, pipes, or box culvert required to handle the runoff, or the Q_d, if ponding is involved.

(5) Compute the discharge velocity (V) in feet per second by the equation $V = Q/A$. If the discharge velocity is greater than the maximum permissible velocity for the outfall (table IX), or the height of the water, in feet, above the top of the culvert inlet is less than $0.222\ V^2$ for CMP or less than $0.017\ V^2$ for concrete pipe or boxes,

drainage systems should be provided. The gravelly and sandy soils with little or no fines (groups GW, GP, SW, and SP) have excellent drainage characteristics. The GMd and SMd groups have fair to poor drainage characteristics, whereas the GMu, GC, SMu, and SC groups may be practically impervious. Soils of the ML, MH, and Pt groups have fair to poor drainage characteristics. All of the other groups have poor drainage characteristics or are practically impervious.

g. Compaction Equipment. The compaction of soils for roads and runways, especially for the latter, requires that a high degree of density be attained at the time of construction in order that detrimental consolidation will not take place under traffic. In addition, the detrimental effects of water are lessened in cases where saturation or near saturation takes place. Processed materials, such as crushed rock, are often used as base courses and such materials require special treatment in compaction. Types of compaction equipment that will usually produce the desired densities are shown in column 12 of figure 61. It may be noted that several types of equipment are listed for some of the soil groups. This is because variations in soil type within a given group may require the use of different equipment. In some cases more than one type of equipment may be necessary to produce the desired densities. Steel-wheeled rollers are recommended for angular materials with limited amounts of fines; crawler-type tractors or rubber-tired rollers for gravels and sand; and sheepsfoot rollers for coarse-grained or fine-grained soils having some cohesive qualities. Rubber-tired rollers are also recommended for final compaction operations for most soils except those of high liquid limit (grou H). Suggested minimum weights of the various types of equipment are shown in note 5 of the table. In column 13 are shown ranges of unit dry weight for soils compacted according to the modified AASHO compaction procedure. These values are included primarily for guidance; design or control of construction should be based on test results.

h. CBR and K Values. The CBR values (par. 42) shown in column 14 of figure 61 give a relative indication of the strength of the various soil groups as used in flexible pavement design. Similarly, values of subgrade modulus k (par. 43) in column 15 are relative indications of strengths from plate-bearing tests as used in rigid-pavement design. When these tests are used for the design of pavements, actual test values should be used for this purpose instead of the approximate values shown in the tabulation.

35. Field Identification Procedures

The Unified Soil Classification System is so arranged that most soils may be classified into at least the three primary groups

(coarse grained, fine grained, and highly organic) by means of visual examination and simple field tests. Classification into the subdivisions can also be made by visual examination with some degree of success. More positive identification may be made by means of laboratory tests on the materials, as described in TM 5–530. The easiest and best way to learn field soil identification procedures is under the guidance of an experienced soils engineer. Lacking this guidance, one should systematically compare the test results which are obtained for soils in each group with the observed behavior of the soil during the process of field identification.

a. Coarse-Grained Soils. Principal items of importance in the field identification (par. 38) of coarse-grained soils are grain size, gradation, and the plasticity characteristics of the fraction passing the No. 40 sieve, if present. Gravel and sand particles can be readily identified with the naked eye. The size corresponding to the No. 200 sieve is about the smallest which can be detected in this fashion. A comparison of the size limits corresponding to the various gravel and sand fractions was given in paragraph 31 and table XII. Gradation and size can be judged by spreading out a representative sample on a flat surface. Well-graded soils show a wide range of grain sizes and a substantial amount of all intermediate particle sizes. Poorly graded soils show grains of essentially one size, or a range of sizes with some intermediate sizes missing. Some idea of the gradation of sands may be obtained by shaking the sample in water in a glass jar and then allowing it to settle out. Approximate gradation is indicated by the separation of the particles in the jar, coarse to fine, from bottom to top. The plasticity characteristics of the fines which may be present in coarse-grained soils are judged by the procedures indicated below for fine-grained soils.

b. Fine-Grained Soils. The field identification procedures are o be performed upon the particles passing a No. 40 sieve (smaller than approximately $\frac{1}{64}$ inch). For field classification purposes, screening of the sample is not intended; simply remove by hand the coarse particles that interfere with the tests. The three tests principally used in the field identification of fine-grained soils are—

(1) Dilatancy or reaction-to-shaking test,
(2) Dry strength or crushing characteristics, and
(3) Toughness or consistency near plastic limit.

These procedures are explained in detail in figure 59 and paragraph 38, together with the range of results of each test which are applicable to soils in each of the fine-grained soil groups.

c. Highly Organic Soils. These soils are readily identified by color (as described in connection with fine-grained soils, par. 38*a*

(1)), odor, and spongy feel, and frequently by fibrous texture. The existence of large amounts of organic matter also may often be detected by a distinctive odor, that of decayed vegetation. The odor is particularly strong in fresh samples, but can be reintensified by heating a wet sample quickly.

36. Field Identification Equipment

a. Practically all the tests to be described in paragraph 38 may be performed with no equipment or accessories other than a small amount of water. However, the accuracy and uniformity of results will be greatly increased by the proper use of certain items of equipment.

b. The following items of equipment will meet most requirements, are available in nearly all engineer units or may be improvised, and are easily transported:

(1) A No. 40 US Standard sieve, perhaps the most useful item of equipment. Any screen with about 40 openings per linear inch could be used, or an approximate separation may be made by sorting the materials by hand. No. 4 and No. 200 sieves are useful for separating gravel, sand, and fines.

(2) Pick and shovel, or set of entrenching tools, for use in obtaining samples. A hand earth auger is useful if samples are desired from depths more than a few feet below the surface.

(3) Spoon, issued as part of mess equipment, to aid in obtaining samples and for use in mixing materials with water to desired consistency.

(4) Combat knife, or engineer pocket knife, for use in obtaining samples and trimming them to the desired size.

(5) Small mixing bowl with a rubber faced pestle for use in pulverizing the fine-grained portion of the soil. Both may be improvised, such as by using canteen cup and wood pestle.

(6) Several sheets of heavy paper for rolling samples.

(7) Pan and heating element for drying samples.

(8) Balances or scales for weighing samples.

37. Factors Considered in Identification of Soils

a. The soil properties which form the basis for the Unified Soil Classification System are: (1) percentages of gravel, sand, and fines, (2) shape of the grain-size distribution curve, and (3) plasticity. These same properties are, therefore, the primary ones to be considered in field identification, but other characteristics observed should also be included in describing the soil, whether the identification is made by field or laboratory methods.

b. Properties normally included in a description of a soil are as follows:

(1) Color.
(2) Grain size.
 (*a*) Estimated maximum grain size.
 (*b*) Estimated percent by weight of fines (material passing No. 200 sieve).
(3) Gradation.
(4) Grain shape.
(5) Plasticity.
(6) Predominant soil type.
(7) Secondary components of soil.
(8) Classification symbol.
(9) Other remarks, such as—
 (*a*) Organic, chemical, or metallic content.
 (*b*) Compactness.
 (*c*) Consistency.
 (*d*) Cohesiveness near plastic limit.
 (*e*) Dry strength.
 (*f*) Source—residual, or transported (aeolian, waterborne, glacial deposit, etc.).

c. An example of a soil description using the sequence and considering the properties referred to above might be as follows:

(1) Dark brown to white.
(2) Coarse-grained soil, maximum particle size 2¾ inches, estimated 60 percent gravel, 35 percent sand, and 5 percent passing No. 200 sieve.
(3) Poorly graded (insufficient fine gravel, gap-graded).
(4) Gravel particles subrounded to rounded.
(5) Nonplastic.
(6) Predominantly gravel.
(7) With considerable sand and small amount of nonplastic fines (silt).
(8) GP.
(9) Slightly calcareous, no dry strength, dense in the undisturbed state.

d. A complete description with the proper classification symbol obviously conveys much more to the reader than the symbol or any other isolated portion of the description used alone.

38. Field Identification Tests

a. Visual Observation. By visual examination it is possible to determine the color, grain size, and grain shape of the coarse-grained portion of a soil, and estimate the grain-size distribution.

To observe these properties, a sample of the material is first dried and then spread on a flat surface.

(1) *Color.* In field soil surveys, color is often helpful in distinguishing between various soil strata. With sufficient preliminary experience with local soils, color may also be useful for identifying soil types. Since the color of a soil varies with its moisture content, the condition of the soil when color is determined must always be recorded. There is generally more contrast in these colors when the soil is moist, with all the colors becoming lighter as the moisture content is reduced. In fine-grained soils, certain dark or drab shades of gray or brown, including almost black colors, indicate organic colloidal matter (OL, OH). In contrast, clean and bright looking colors, including medium and light gray, olive green, brown, red, yellow, and white, are generally associated with inorganic soils. Soil color may also indicate the presence of certain chemicals. Red, yellow, and yellowish brown soil colors may be a result of the presence of iron oxides. White to pinkish colors may indicate presence of considerable silica, calcium carbonate, or aluminum compounds in some cases. Grayish blue and gray and yellow mottled colors, frequently indicate poor drainage.

(2) *Grain size.* The maximum particle size should always be estimated for each sample considered, thereby establishing the upper limit of the grain-size-distribution curve for that sample. To aid in determining something about the lower limit of the grain-size distribution, it is useful to know that the naked eye can normally distinguish the individual grains of soil down to about 0.05 millimeter. This means that all of the particles in the gravel and sand ranges are visible as such to the naked eye. Most of the silt particles and all of the clay particles are smaller than this size and are therefore indistinguishable to the naked eye. Material smaller than 0.05 millimeter will pass the No. 200 sieve.

(3) *Approximate grain-size distribution.* The laboratory mechanical analysis must be performed whenever the grain size distribution of a soil sample must be determined accurately. However, an approximation of the grain size distribution can be determined by visual inspection. The best method of observing a material for such a determination without using laboratory equipment is to spread a portion of the dry sample on a flat surface and then, using the hands or a piece of paper,

attempt to separate the material into its various grain size components. By this method, the gravel particles and some of the sand particles can be separated from the remainder. This will at least give an opportunity to estimate whether the total sample is to be considered coarse-grained or fine-grained, depending on whether or not more than 50 percent of the material would pass the No. 200 sieve. (Percentage values refer to the dry weight of the soil fractions indicated as compared to the dry weight of the original sample.) If the material is believed to be coarse grained, two other criteria are to be considered. The first is whether less than 5 percent passes the No. 200 sieve, and the second is whether the fines interfere with the soil's free-draining properties. If both these criteria can be satisfied and there appears to be a good representation of all grain sizes from largest to smallest, without an excessive amount or a deficiency of any one size, the material may be said to be well graded (GW or SW). If any intermediate sizes appear to be missing, or if there is too much of any one size, then the material is poorly graded (GP or SP). Using the No. 4 and No. 200 sieves, the sample may be separated into the three main fractions; that is, gravel, sand, and fines. However, if there is a considerable quantity of fines, particularly clay particles, separation of the fines can only be readily accomplished by washing them through the No. 200 sieve. In such cases, the percentage of fines is determined by comparing the dry weight of the original sample with that retained on the No. 200 sieve after washing. The difference between these two is the weight of the fines lost in the washing process. For determination of plasticity, only that portion of the soil which will pass through a No. 40 sieve should be used. Estimating the grain-size distribution of a sample without using any equipment at all is probably the most difficult part of field identification and obviously places great importance on the experience of the individual making the estimate. A better approximation of the relative proportions of the components of the finer soil fraction may sometimes be obtained by shaking a portion of this sample into a jar of water and then allowing the material to settle to the bottom. The material will settle in layers with the gravel and coarse sand particles settling out almost immediately, the fine sand particles within a minute, the silt particles requiring as much as an hour, and the clay particles remaining in suspension

much longer. In using this method, it should be kept in mind that the gravel and sand will settle into a much more dense formation than will either the silt or clay.

(4) *Properties of undisturbed soils.* A complete description of a soil should include prominent characteristics of the undisturbed materials. The aggregate properties of sand and gravel are described qualitatively by the terms "loose," "medium," and "dense," whereas those of clays are described by "hard," "stiff," "medium," and "soft." These characteristics are usually evaluated by the soil-survey foreman on the basis of several factors, including the relative ease or difficulty of advancing the drilling and sampling tools and the consistency of the sample. In soils that are described as "soft" it should also be indicated whether the material is loose and compressible, as in an area under cultivation, or spongy or elastic, as in highly organic soils.

b. Breaking or Dry-Strength Test. The breaking test is performed only on the material passing the No. 40 sieve. This test, as well as the roll test and the ribbon test, is used to measure the cohesive and plasticity characteristics of the soil. The test is normally made on a small pat of soil about ½ inch thick and about 1½ inches in diameter. The pat is prepared by molding a portion of the soil in the wet plastic state into the size and shape desired and then allowing it to air dry. The soil sample must not be oven dried. Samples may be tested for dry strength in their natural condition as they are found in the field, but too much reliance must not be given to such tests because of the variations that exist in the drying conditions under field circumstances. Such a test may be used as an approximation, however, and verified later by a carefully prepared sample. After the prepared sample is thoroughly dry, the tester should attempt to break it by using the thumb and forefingers of both hands (fig. 62). If it can be broken, he should try to powder it by rubbing the two particles together between the thumb and fingers of one hand (fig. 63). Typical reactions obtained in this test from various types of soils are described below.

(1) Very highly plastic soils (CH), very high dry strength. Samples cannot be broken or powdered by use of finger pressure.

(2) Highly plastic soils (CH), high dry strength. Samples can be broken with great effort, but cannot be powdered.

(3) Medium plastic soils (CL), medium dry strength. Samples can be broken and powdered with some effort.

(4) Slightly plastic soils (ML, MH, or CL), low dry strength. Samples can be broken quite easily and powdered readily.

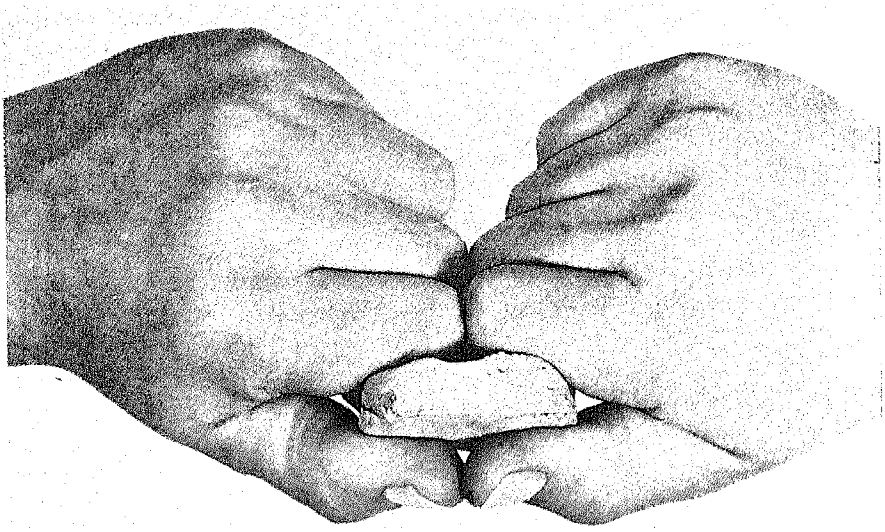


Figure 62. Breaking dry sample.

Figure 63. Powdering dry sample.

(5) Nonplastic soils (ML or MH), very little or no dry strength. Samples crumble and powder on being picked up in the hands.

Note. This is one of the best tests for distinguishing between plastic clays and nonplastic silts or fine sands, but a word of caution is appropriate. Dry pats of highly plastic clays quite often display shrinkage cracks. To break the sample along one of these cracks will not give an indication of the true dry strength. It is important to distinguish between a break along such a crack, and a clean, fresh break that indicates the true dry strength of the soil.

c. Roll or Thread Test. The roll or thread test is performed only on the material passing the No. 40 sieve. The soil sample used in this test is prepared by adding water until the moisture content is such that the sample may be easily molded without sticking to the fingers. This is sometimes referred to as being just below the "sticky limit." Using a sheet of heavy paper as a surface, this sample is rolled rather rapidly into a thread approximately 1/8 inch in diameter (fig. 64). Materials which cannot be rolled in this manner are nonplastic, or have very low plastcity. If a soil can be rolled into such a thread at some moisture content, it is said to have some plasticity. The number of times that the thread may be lumped together and the rolling process repeated without crumbling and breaking is a measure of the degree of plasticity of the soil. After reaching the plastic limit (when the soil begins the crumble when rolled) the degree of plasticity may be determined as follows:

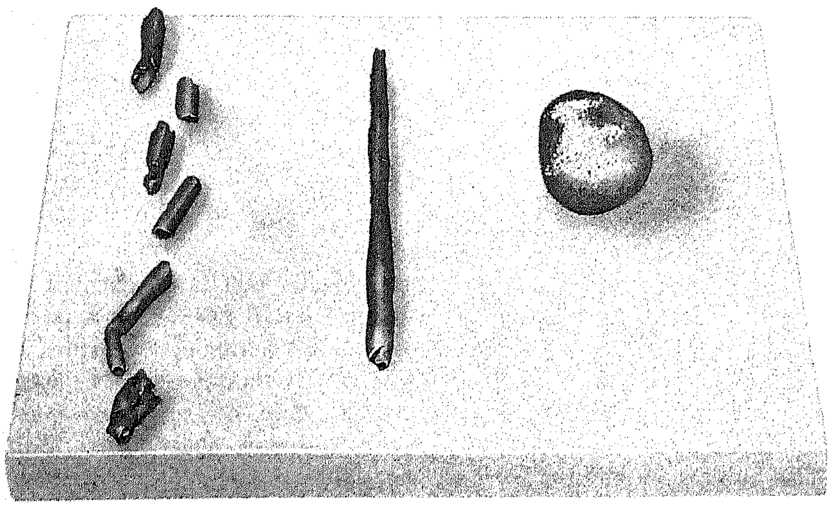


Figure 64. Roll or thread test.

(1) High plasticity (CH)—the soil may be remolded into a ball and the ball deformed under extreme pressure by the fingers without cracking or crumbling.

(2) Medium plasticity (CL)—the soil may be remolded into a ball, but the ball will crack and easily crumble under pressure of the fingers.

(3) Low plasticity (CL, ML, or MH)—the soil cannot be lumped together into a ball without completely breaking up.

(4) Organic materials (OL or OH)—soils containing organic materials or mica particles will form soft spongy threads or balls when remolded.

(5) Nonplastic soils (ML or MH)—these soils cannot be rolled into a thread at any moisture content.

From this test the cohesiveness of the material near the plastic limit may also be described as weak, firm, or tough. The higher the position of a soil on the plasticity chart, the stiffer the threads as they dry out and the tougher the lumps if the soil is remolded after rolling.

d. Ribbon Test. The ribbon test is performed only on the material passing the No. 40 sieve. The sample prepared for use in this test should have a moisture content that is slightly below the "sticky limit" (*c* above). This sample is formed into a roll of soil about ½ to ¾ inch in diameter and about 3 to 5 inches long. This roll is placed in the palm of the hand. Starting at one end, the roll is then flattened to form a ribbon ⅛ to ¼ inch thick, by squeezing it between the thumb and forefinger (fig. 65). The sample should be handled carefully to form the maximum length of ribbon that can be supported by the cohesive properties of the soil. If the soil sample holds together for a length of 8 to 10 inches without breaking, the material is then considered highly plastic. If the soil cannot be ribboned, it is nonplastic (ML or MH). If it can be ribboned only with difficulty into short lengths, the soil is considered to have low plasticity (CL). The roll test and the ribbon test complement each other in giving a clearer picture of the degree of plasticity of soil.

e. Wet Shaking Test. The wet shaking test is performed only on the material passing the No. 40 sieve. In preparing a portion of the sample for use in this test, enough material to form a ball of material about ¾ inch in diameter is moistened with water. This sample should be just wet enough that the soil will not stick to the fingers upon remolding, or just below the "sticky limit." The sample is then placed in the palm of the hand and shaken vigorously. This is usually done by jarring the hand on the table or some other firm object, or by jarring it against the other hand.

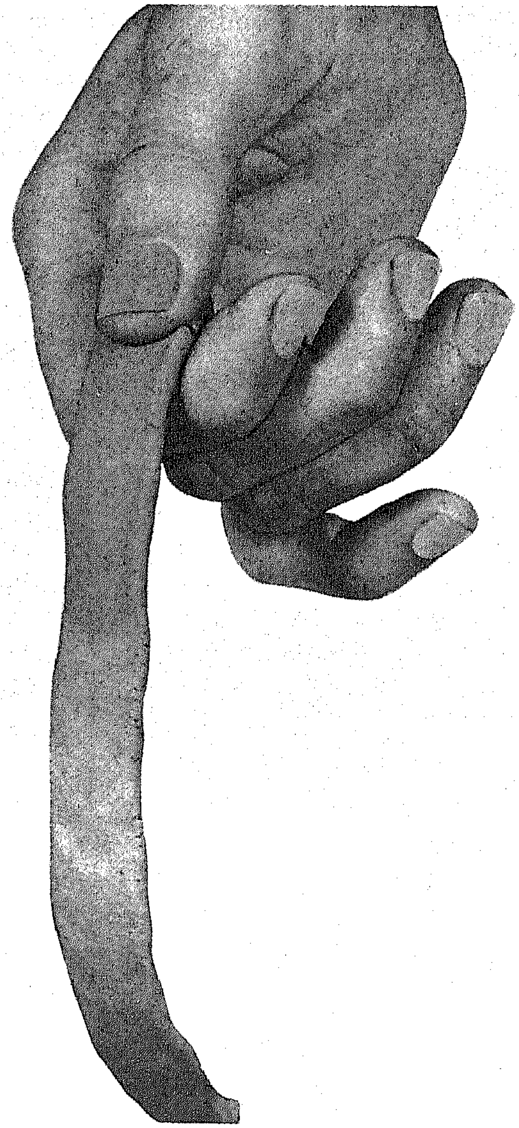


Figure 65. Ribbon test. Sample shown is a highly plastic clay (CH) and has formed a ribbon about 9 inches long and ⅛ to ¼ inch thick.

The soil is said to have given a reaction to this test when, on shaking, water comes to the surface of the sample, producing a smooth, shiny appearance. This appearance is frequently described as "livery" (fig. 66). Then, upon squeezing the sample between the thumb and forefinger of the other hand, this surface

water will quickly disappear, the surface will become dull (fig. 67), the material will become firm, resisting deformation, and cracks will occur as pressure is continued, with the sample finally crumbling like a brittle material (fig. 68). The vibration caused by the shaking of the soil sample tends to reorient the soil grains, decrease the voids, and force water from within these voids to the surface. Pressing the sample between the fingers tends to disarrange the soil grains and increase the voids space, and the water is drawn into the soil again. If the water content is still adequate, shaking the broken pieces will cause them to liquefy again and flow together, and the complete cycle may be repeated. This process can occur only when the soil grains are noncohesive in character. Very fine sands and silts fall into this category and are readily identified by the wet shaking test. Since it is rare that fine sands and silts occur without some amount of clay mixed with them, there are varying degrees of reaction to this test. Even a small amount of clay will tend to greatly retard this reaction. Some of the descriptive terms applied to the different rates of reaction to this test are as follows:

(1) *Sudden or rapid.* A rapid reaction to the shaking test is typical of nonplastic, fine sands and silts (fig. 69).

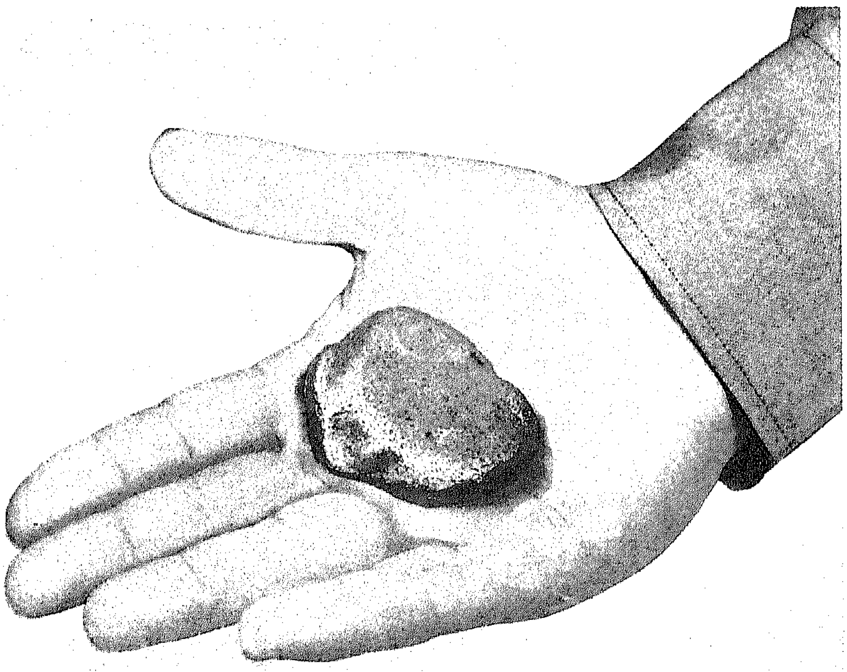


Figure 66. Wet shaking test; sample in "livery" condition.

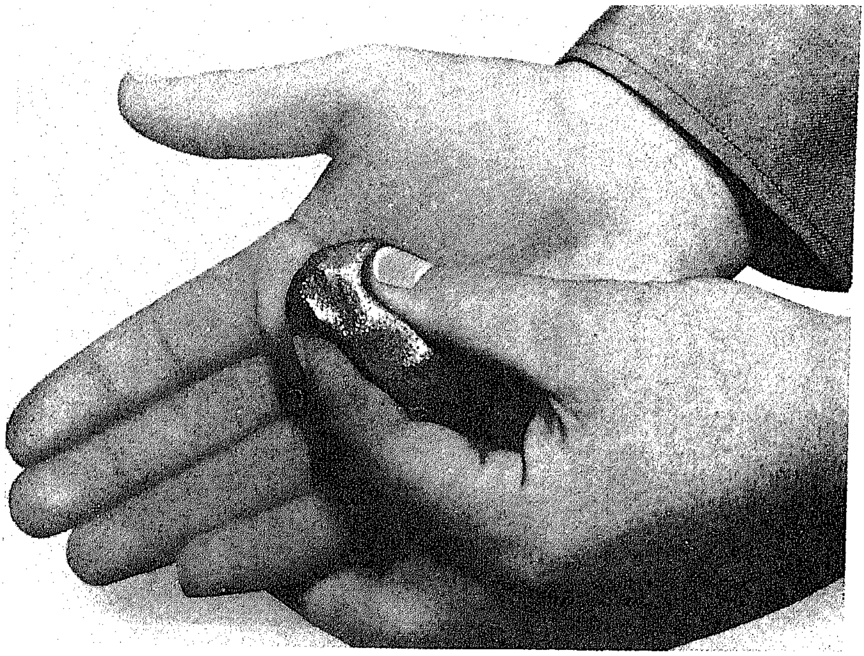


Figure 67. Wet shaking test; sample exhibiting surface dulling upon being squeezed.

A material known as rock flour which falls into the silt size ranges also gives this type of reaction.

(2) *Sluggish or slow.* A sluggish reaction indicates slight plasticity such as might be found from a test of some organic or inorganic silts, or silts containing a small amount of clay.

(3) *No reaction.* Obtaining no reaction at all to this test does not indicate a complete absence of silt or fine sand. Even a slight content of colloidal clay will impart some plasticity and slow up materially the reaction to the shaking test. Extremely slow or no reaction is typical of all inorganic clays and of the highly plastic organic clays.

f. Odor Test. Organic soils of the OL and OH groups usually have a distinctive, musty, slightly offensive odor which, with experience, can be used as an aid in their identification. This odor is especially apparent from fresh samples. It is gradually reduced by exposure to air, but can again be made more pronounced by heating a wet sample. Organic soils are undesirable as foundation or base-course material and are usually removed from the construction site.

Figure 68. Wet shaking test; sample in crumbling condition.

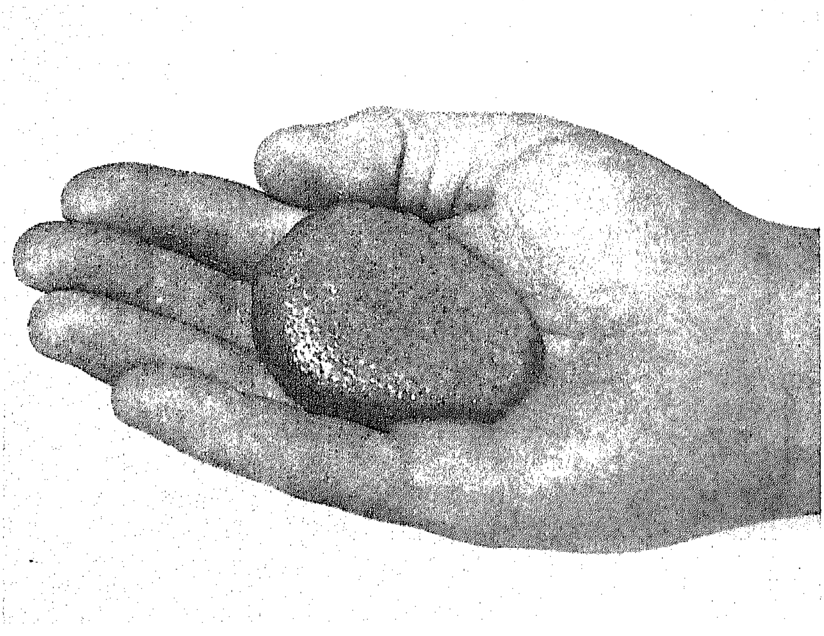


Figure 69. Wet shaking test; sample showing rapid reaction to shaking.

g. Bite or Grit Test. The bite or grit test is a quick and useful method of identifying sand, silt, or clay. In this test a small pinch of the soil material is ground lightly between the teeth. The soils are identified as follows:

(1) *Sandy soils.* The sharp hard particles of sand will grate very harshly between the teeth and will be highly objectionable. This is true even of fine sand.

(2) *Silty soils.* The silt grains are so much smaller than sand grains that they do not feel nearly so harsh between the teeth, and are not particularly objectionable, although their presence is still easily detected.

(3) *Clayey soils.* The clay grains are not at all gritty, but feel smooth and powdery like flour between the teeth. Dry lumps of clayey soils will stick when lightly touched with the tongue.

h. Slaking Test. The slaking test is used to assist in determining the quality of certain soft shales and other soft "rock like" materials. The test is performed by placing the soil in the sun or in an oven to dry, and then allowing it to soak in water for at least 24 hours. The strength of the soil is then examined. Certain types of shale will completely disintegrate, losing all strength like the soaked sample in the jar shown in figure 70. Other mate-

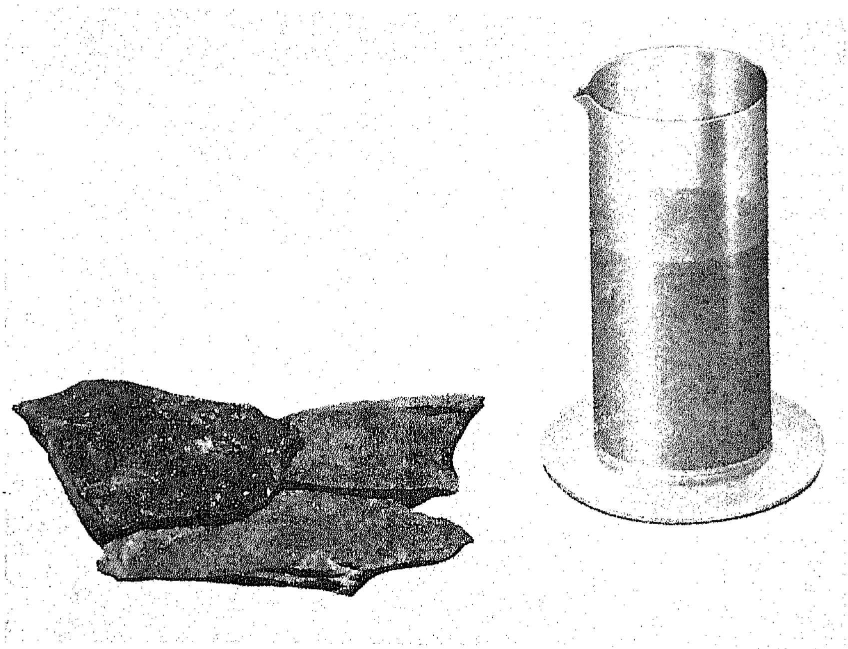


Figure 70. Slaking test. Samples shown before and after soaking. Soaked sample in jar has disintegrated.

rials that appear to be durable rocks may be crumbled and readily broken by hand after such soaking. Materials which show a considerable reduction in strength upon soaking are undesirable for use as base course materials.

i. Acid Test. The acid test is used to determine the presence of calcium carbonate and is performed by placing a few drops of hydrochloric acid on a piece of the soil (fig. 71). A fizzing reaction (effervescence) indicates the presence of calcium carbonate, and the degree of reaction gives an indication of the concentration. Calcium carbonate is normally desirable in a soil because of the cementing action it provides to add to the stability. Since this cementation is normally developed only after a considerable curing period, it cannot be counted upon for strength in much military construction. The primary use for this test is, therefore, to permit better evaluation of what appears to be abnormally high strength values of fine-grained soils which are tested in-place, where this cementation property may exert considerable influence.

j. Shine Test. The shine test is another means of measuring the plasticity characteristics of clays. A slightly moist or dry

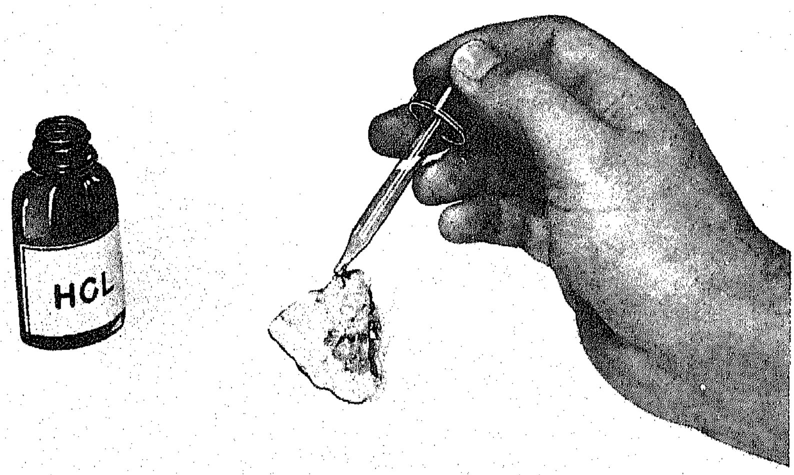


Figure 71. Acid test.

piece of highly plastic clay will give a definite shine when rubbed with a fingernail, a pocket-knife blade, or any smooth metal surface. On the other hand, a piece of lean clay will not display any shine, but will remain dull.

k. Feel Test. The feel test is a general purpose test, and one that requires considerable experience and practice in its use before reliable results can be obtained. Some of the following characteristics can be readily estimated by proper use of this test:

(1) *Moisture content.* The natural moisture content of a soil is a value as an indicator of the drainage characteristics, nearness to water table, or other factors which may affect this property. A piece of undisturbed soil is tested by squeezing it between the thumb and forefinger to determine its consistency. The consistency is described by such terms as "hard," "stiff," "brittle," "friable," "sticky," "plastic," or "soft." The soil is then remolded by working it in the hands, and changes, if any, are observed. By this test the natural water content is estimated relative to the liquid or plastic limit of the soil. Clays which turn almost liquid on remolding are probably near or above the liquid limit. If the clay remains stiff and crumbles upon being remolded, the natural water content is below the plastic limit.

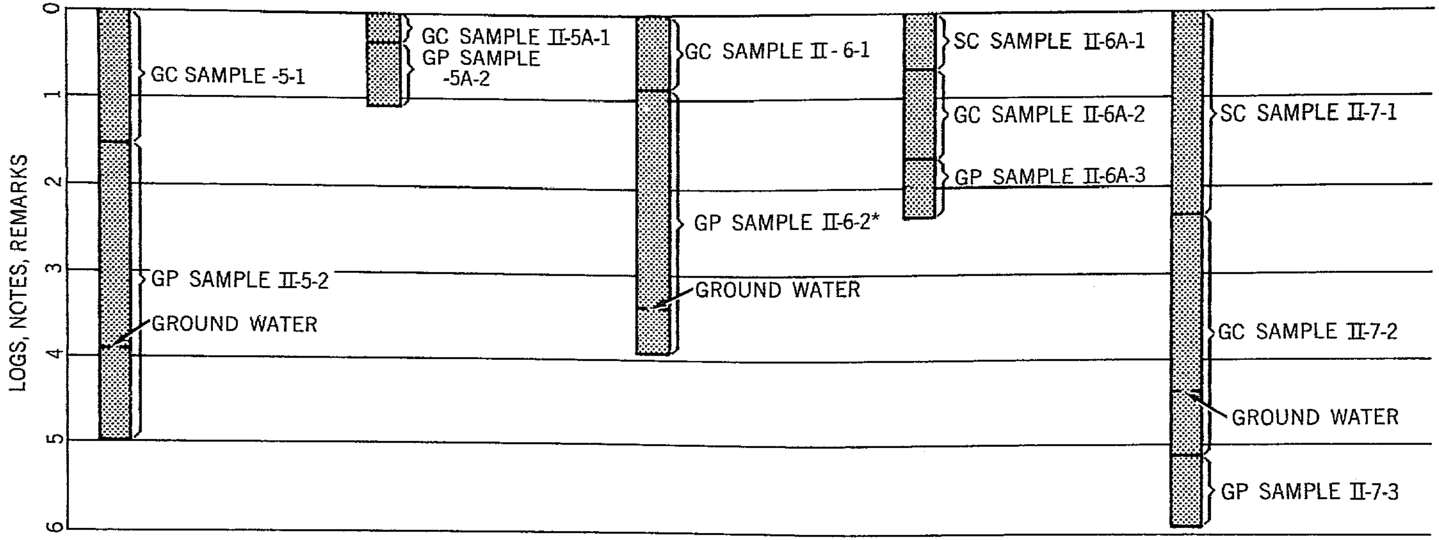
(2) *Texture.* The term texture, as applied to the fine-grained portion of a soil, refers to the degree of fineness and uniformity. It is described by such expressions as "floury," "smooth," "gritty," or "sharp," depending on the sensation produced by rubbing the soil between the fingers. Sensitivity to this sensation may be increased by rubbing some of the material on a more tender skin area such as the inside of the wrist. Fine sand will feel gritty. Typical dry silts will dust readily, and feel relatively soft and silky to the touch. Clay soils are powdered only with difficulty but become smooth and gritless like flour.

39. Water Table

The elevation of the water table is determined during the soil survey by observing the level at which free water stands in the test hole, usually an auger boring. To get an accurate determination, holes should be covered, after being dug, and inspected 24 hours later, in order to allow the water to reach its maximum level.

40. Plotting and Using the Soil Profile

In engineering, the soil profile is defined as a graphical representation of a vertical cross section of the soil layers from the



* SAMPLE II-6-2 MEANS THAT THIS IS THE SECOND SAMPLE IN TEST HOLE NO. 6 LINE II. GP REFERS TO SOIL GROUP IN THE UNIFIED SOIL CLASSIFICATION SYSTEM.

Figure 72. Consolidated field data of a soil survey.

surface of the earth downward. Where heavy structures are involved, the soil profile may be determined to a depth of 30 feet or more below the surface. For roads and airfields, the depth of the profile is generally much less, as described in paragraph 28*a*.

a. A detailed field log is kept of each auger boring or test pit made during the soil survey. When the survey has been completed, the information contained in the separate logs is consolidated, as shown in figure 72. In addition to the information shown, it is desirable to include the natural water content of fine-grained soils along the side of each log, when this information is available. Other descriptive abbreviations may be used, as deemed appropriate.

b. The soil profile is drawn from the consolidated field data. It shows the location of test holes, profile of the natural ground to scale, location of any ledge rock encountered, field identification of each soil type, thickness of each soil stratum, profile of the water table, and the profile of the finished grade line. In some cases it may be possible to show on the profile the results of laboratory tests on significant soil strata; more frequently this information is given in a separate report. A soil profile based

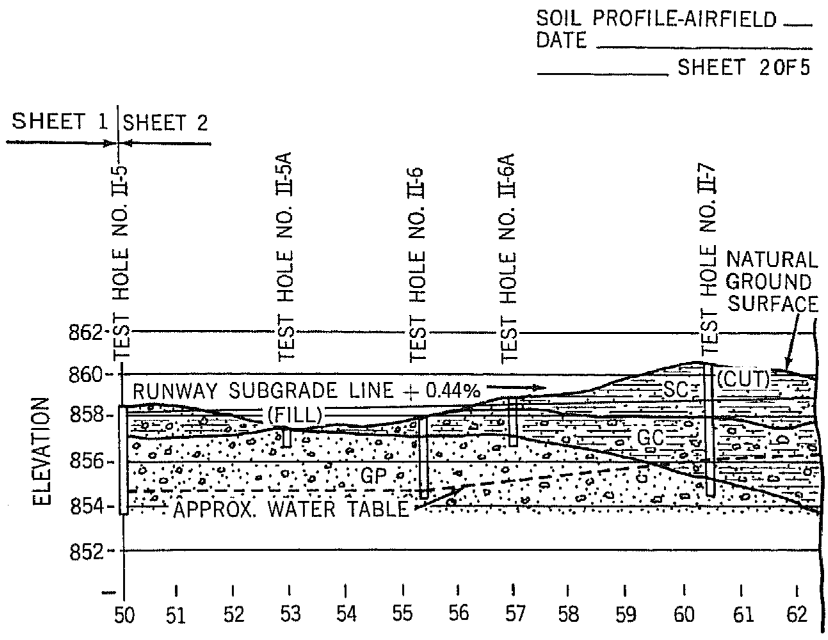


Figure 73. Section of a typical soil-profile sheet.

on the field data of figure 72 is shown in figure 73. Both figures represent part of an airfield soil survey, but the same procedure is used to record the results of soil surveys on road locations.

c. As shown in figures 72 and 73, the soil group of the Unified Soil Classification System is shown on the log or profile by letter designation, such as GP. As an alternative the hatching symbols shown in column (4), figure 61, may be used. In certain special instances the use of color to delineate soil types on maps and drawings is desirable. A suggested color scheme to show the major soil groups is shown in column (5), figure 61.

d. The soil profile has many practical uses in the location, design, and construction of roads and airfields. It has a great influence on the location of the grade line, which should be placed so that full advantage is taken of the best soils which are available at the site. The profile will show whether soils to be excavated are suitable for use in embankments, or if borrow soils will be required. It may show the existence of undesirable conditions, such as peat or muck, or ledge rock at or close to the surface, which will require special construction measures. It will aid in the planning of drainage facilities so that advantage may be taken of the presence of well-draining soils. It may indicate that special drainage installations will be needed for soils which are more difficult to drain, particularly in areas where the water table is high. Considerations relative to capillarity and frost action may be particularly important when frost-susceptible soils are shown on the profile.

41. Optimum Moisture Content and Other Compaction Considerations

a. Requirements. The density to which a soil may be compacted with a given compactive effort varies with the moisture content. The moisture at which the greatest dry density (dry unit weight) is obtained with a given compactive effort is termed *optimum moisture.* In runway-pavement construction it is desirable to compact subgrade fill, upper zones of subgrades in most instances, and base-course materials to as great a density as is practicable, in order to obtain greater stability and prevent detrimental settlements. The laboratry compaction test used for field control should be one which will indicate the maximum dry unit weight that can be obtained practicably in the field. In the majority of cases, the modified AASHO (American Association of State Highway Officials) compaction procedure as described in TM 5–530 is satisfactory for control purposes.

b. Laboratory Test Procedures. Laboratory test procedures for

determining the moisture-density relationships of soils are explained fully in TM 5–530. In general, the procedure consists of mixing several samples of the same soil at various moisture contents and then compacting them with the same compactive effort. The dry unit weights of the compacted samples are determined and plotted against moisture content. The moisture content which produces the greatest unit weight (maximum density) is the optimum moisture content. Information on the theory of compaction and related items is given in TM 5–541.

c. Tabular Values. In column (13), figure 61, are shown ranges of dry unit weight for the various soil groups, when compacted according to the modified AASHO compaction procedure. These values are included primarily for guidance; design or control of construction should be based on test results.

d. Field Control of Compaction. Laboratory analysis establishes an approximation of the optimum moisture content and the minimum density specifications for the soil which is being compacted in the field. Specifications generally require the attainment of a certain minimum density in the field. For example, a density of at least 95 percent modified AASHO maximum density may be required. Compaction with the moisture content of the soil at or near optimum is generally the most economical and satisfactory procedure. The field compaction procedure is controlled by making field measurements of density and moisture content as rolling proceeds. Densities obtained are compared with minimum densities established for the project. Moisture contents are compared, generally, with the optimum moisture content to see that compaction is taking place within the desired range of moisture content or to permit adjustment if necessary. Several methods of determining densities and moisture contents in the field are described in TM 5–530. Methods described in that manual are density and water-constant determinations by use of chunk samples, the 2-inch drive sampler, the 3-inch drive method, and the oil method. The field-in-place California Bearing Ratio (CBR) test (par. 42) can be used as a construction control test on earthwork, but should be limited to well-graded base courses of hard rock with a plasticity index of less than 5 and plastic subgrade in emergency construction only. Where plastic subgrades are constructed for an anticipated life of 6 months or longer, a family of compaction and CBR curves should be prepared in the laboratory giving moisture-density-CBR relationships. During construction all tests should be field densities; then the moisture content and dry unit weight determined in an individual field test can be plotted on the family of curves and will indicate the CBR values.

42. California Bearing Ratio

a. Description. The California Bearing Ratio test is a measure of the relative bearing capacity of soils and base materials. The test was originally developed as an empirical method for designing flexible highway pavements, and the resulting data has been extended for use in airfield pavement design. The CBR is expressed as a percentage of the unit load required to force a 3-square-inch piston into the soil, divided by the unit load required to force the same size piston the same depth into a standard sample of compacted crushed rock. The CBR used in design is the 0.1- or 0.2-inch penetration value, whichever is greater. For designing the thickness of various base courses, the CBR should be determined for all subgrade soils and base-course materials.

b. Test Procedure. The CBR is used in conjunction with curves for designing flexible pavements. The test procedure for determining the CBR to be used in the design of a specific airfield consists of two principal steps: first, the preparation of soil test specimens, and second, the performance of the penetration test on the prepared soil samples.

(1) A standardized procedure has been established for the penetration portion of the test and fixed procedures for the preparation of test specimens for various types of soils have also been established. The test specimens are prepared so as to duplicate the soil conditions existing (or expected to exist later) in the field. For airfields, the method of preparing test specimens and the number of specimens to be tested depend upon the type of airfield, the soils encountered at the site, and other factors.

(2) In general, for design purposes, the soil should be tested in the laboratory at a density comparable to that to be attained during construction. In cases where moisture conditions are favorable and the subgrade does not accumulate so much moisture it approaches saturation, samples should be tested at a moisture content approximating conditions expected during the time the road or airfield is to be used. Where doubt exists as to whether the moisture and density tests are giving adequate control, evaluative CBR tests may be required. Undisturbed samples are taken in 6-inch diameter molds, subjected to the 4-day soaking test, and tested for CBR in accordance with prescribed procedures. Taking undisturbed samples requires considerable work and in-place CBR tests should be used instead of undisturbed samples where possible. The field-in-place CBR test may be used under any one of the following conditions:

(a) When the layer of saturation (percentage of voids filled with water) is 80 percent or greater.

(b) When the material is coarse grained and cohesionless so that it is not affected by changes in water content.

(c) When the material has been in place for about 3 years or more.

(3) Complete instructions for the performance of CBR tests on laboratory-compacted samples, on undisturbed samples in the CBR mold, and in the field on the soil in place (field CBR) are contained in TM 5–530.

c. *Tabular Values.* The field CBR values shown in column (14), figure 61, give an indication of the relative strength of the various soil groups in relation to the design of flexible pavements. In expedient situations, these approximate values may be used for design. In cases where time and facilities are available, design should be based upon actual test results.

43. Subgrade Modulus

In column (15), figure 61, are given typical values of the subgrade modulus (modulus of subgrade reaction) K. This value is needed in the design of rigid pavements for airfields. It can be determined by the performance of a field plate bearing test, as described in TM 5–541. The test is expensive and time-consuming, and requires considerable experience in the interpretation of test results. For these reasons, it is seldom performed in a theater of operations. For design purposes, values taken from the table will generally be satisfactory in theater-of-operations situations. The subgrade modulus is defined as the reaction of the subgrade per unit of area unit of deformation. Its units are pounds per square inch per inch of deformation, or pounds per cubic inch.

CHAPTER 6
PREPARATION OF SUBGRADES

44. Basic Procedures

a. Introduction. The subgrade is the foundation which supports any load placed on the surface of a runway, taxiway, hardstand, or road. A satisfactory road or runway demands a strong subgrade to prevent deformation of the surface (fig. 74). The strength of the subgrade determines the thickness of the pavement and base. In general, compaction increases the strength of a subgrade soil and provides a more uniform foundation on which the base and pavement may be constructed. The additional strength obtained by compaction permits the use of a thinner pavement and base than would otherwise be required. For certain soils, however, compaction procedures may produce decreases in strength. These require special treatment, as described in paragraph 49.

b. Laboratory Tests. Before construction starts, subgrade soils are subjected to a soil survey (pars. 24–27) and to various tests

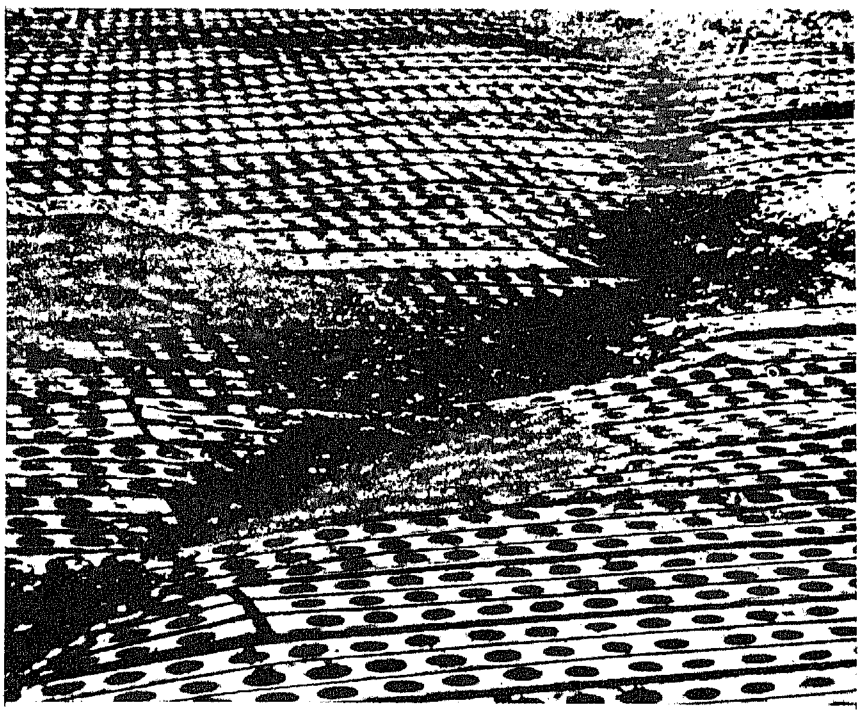


Figure 74. Weak subgrade areas result in deformed surface.

which may include mechanical-analysis, liquid-limit, plastic-limit, in-place moisture-content, in-place-density, compaction, and CBR tests. Although these tests are performed primarily for design purposes, the results should be available for field use when construction begins. Some of the tests described in paragraph 38 are also of value in predicting soil behavior.

c. Practical Tests. Practical tests of subgrade soil include observation of the behavior of the subgrade under loaded trucks or tests of trial sections of subgrade covered with the proposed base. For a detailed explanation of testing procedures see TM 5–530.

d. Advantages of Compaction. Because of the advantages it gives, the compaction of soils has become a standard procedure in the construction of earth structures, particularly embankments, earth-fill dams, subgrades, and bases for highway and airport pavements. There is no single construction process which can be applied to natural soils which produces so marked a change in their physical properties at so low a cost as compaction, when it is properly controlled to produce the desired results. Principal soil properties which are affected by compaction include settlement, shearing resistance, movement of water, and volume change. Compaction does not improve the desirable properties of all soils in the same degree, and in certain cases the engineer must give careful consideration to the effect of compaction upon the various properties listed. For example, with certain soils the desire to hold volume change to a minimum may be more important than just an increase in shearing resistance.

(1) *Minimizing of settlement.* One of the principal advantages resulting from the compaction of soils used in embankments is that it reduces to a minimum any settlement which might be caused by the consolidation of the soil within the body of the embankment. This is true because compaction and consolidation both bring about a closer arrangement of soil particles. Artificial densification by compaction will prevent later consolidation and settlement of an embankment, but this does not necessarily mean that the embankment will be completely free from settlement, since its weight may cause consolidation of compressible soil layers which may form the embankment foundation.

(2) *Increase in shearing resistance.* Increasing density by compaction increases shearing resistance. This effect is highly desirable, since it may make possible the use of a thinner pavement structure over a compacted subgrade or the use of steeper side slopes for an embankment than would otherwise be possible. Other things being the

same, the shearing resistance of a given soil increases with an increase in density. In addition to density, shearing resistance depends upon water content. For the same density the highest strengths are frequently obtained by the use of greater compactive efforts and with water contents somewhat below optimum moisture content (par. 41). Large-scale experiments conducted by the Corps of Engineers indicate that the unconfined compressive strength of a clayey sand could be doubled by compaction, within the range of practical field compaction procedures.

(3) *Effects on water movement.* When soil particles are forced together by compaction, both the amount of voids contained in the soil mass and the size of the individual void spaces are reduced. This change in voids has an obvious effect upon the movement of water through the soil. One effect is to reduce the permeability, thus reducing the seepage of water. Somewhat similarly, if the compaction is accomplished with proper moisture control, the movement of capillary water is minimized, thus reducing the tendency for the soil to take up water and suffer later reductions in shearing resistance.

(4) *Volume change.* Volume change (shrinkage and swelling) is an important soil property which is particularly critical when soils are used as subgrades for roads and airport pavements. Generally speaking, volume change is not a great matter of concern in relation to compaction, except for clay soils. Compaction has a marked influence on the volume change of clay soils. For these soils, the greater the density, the greater the potential volume change, unless the soil is restrained. An expansive clay soil should be compacted at a moisture content higher than the laboratory or field optimum and to a density at which swelling will be a minimum. Similarly, the soil should be compacted so that shrinkage will be a minimum. Although the conditions corresponding to minimum swell and minimum shrinkage may not be exactly the same, soils in which volume change is a factor generally may be compacted so that these effects are minimized. The effect of swell on bearing capacity is important and is recognized by the standard method (TM 5–530) used by the Corps of Engineers in preparing samples for the California Bearing Ratio test.

45. Subgrade-Construction Equipment

Numerous types of equipment, some of which are described in

TM 5–252, are used in subgrade construction. Some types work well in one material but are entirely inadequate in others. It may prove necessary to use several types of equipment in conjunction with each other on one project. The equipment must be capable of handling the material needed for an entire depth of lift in one operation. The description of equipment that follows should help in determining whether the construction forces have the right amount and types for the material to be handled on a particular job.

a. Rotary-Tiller Mixer.

(1) The rotary-tiller mixer is used to pulverize or mix materials in place. It is capable of mixing different size aggregates, including gravel, crushed stone, sand, clay, and broken-up bituminous paving, with any suitable binder material, to build roads, runways, or parking areas. The mixing action which takes place in the machine is shown in figure 75.

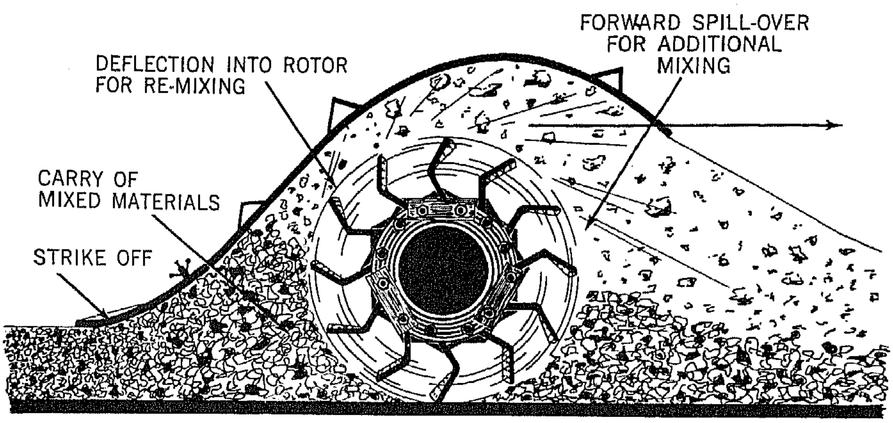


Figure 75. Mixing action in a rotary-tiller mixer.

(2) In making any cut through a knoll or hill, the gradeline of the roadbed shears diagonally across various soil layers, often in rapid succession. Each layer will usually react differently to frost action, to moisture variation, and to expansion by summer heat. So great is the variation of subgrade movements due to such effects that even a concrete pavement laid upon such a variable subgrade will display its greatest tendency to crack or fault at the mouth of a cut. To counteract this undesirable tendency, subgrade soils should be mixed or blended with a rotary-tiller mixer. This destroys the separate identities of the

various soil layers, establishes a uniformity of subsoil properties, and minimizes the dangers of differential frost heaving or irregular heat expansion.

(3) The subgrade should be stabilized by mixing and blending the native soil to a depth of from 4 to 12 inches, as may be required. If the ground is hard, it should be scarified full depth before mixing with the rotary-tiller mixer.

b. Tandem Gang Disks. Tandem gang disks are good for all types of material except coarse stone. They are very good for preparing foundations and breaking down chunks. They are also good for blending materials and uniformly mixing moisture into the material. The weighting of the gang disks to penetrate the entire lift of loose material to be compacted is of prime importance.

c. Gang Plow. The gang plow may be used for all types of materials. It mixes moisture uniformly. It blends materials well but will not break down chunks.

d. Spring-Tooth Harrow. The spring-tooth harrow is used in low-plasticity materials only. Weighting of the harrow is important.

e. Heavy-Duty Cultivator. The heavy-duty cultivator is good for low-plasticity materials.

f. Grader. The grader blade is extensively used both for blending subgrade soils and for aerating them to reduce moisture content.

g. Water Truck. The pressure-tank water truck gives a more uniform flow of water, although the gravity-type tank is in most general use. The controls for the spray bars should be in the cab. Spray bars should not leak. To insure uniform water coverage the truck should be moving at uniform speed from the time immediately before the sprays are turned on until after the sprays are turned off.

h. Heavy Pneumatic-Tired Roller. This roller is designed so that the weight can be varied to apply the desired compactive effort. Rollers with capacities up to 50 tons usually have four wheels and tires designed for 90 psi inflation. They can be obtained with tires designed for inflation pressures up to 150 psi. As a rule, the higher the tire pressure, the greater the contact pressures, and consequently, the greater the compactive effort obtained. The lift thickness should not exceed 6 inches after compaction where compaction to 90 percent or more of modified AASHO density is required. For hasty construction thicker lifts can be used—up to 18 inches for the 50-ton pneumatic-tired roller under favorable conditions and 12 inches for the 13-wheel roller. When a large rubber-tired roller is to be used, care should be exercised to insure that the moisture content of cohesive materials is

low enough so that excessive pore pressures do not occur. Weaving or springing of the soil under the roller is evidence that pore pressures are developing. The large rubber-tired roller can be used for all types of material. It is used especially for final compaction of the upper 6 inches of subgrade, for subbases, and for base courses. These rollers are very good for obtaining a high degree of compaction.

i. Sheepsfoot Roller. When using a sheepsfoot roller the lift thickness should not exceed 6 inches after compaction where compaction to 90 percent or more of modified AASHO is required. For hasty construction lifts up to 9 inches may be used. A uniform density can generally be obtained throughout the full depth of the lift if the material is loose and workable enough to permit the sheepsfoot roller to penetrate into the layer for the first passes. This produces compaction from the bottom up. Material that becomes compacted by the wheels of equipment during pulverizing, wetting, blending, and mixing should, therefore, be thoroughly loosened before compaction operations are begun. This will also insure uniformity of the mixture. The same amount of rolling will generally produce increased densities as the depth of the lift is decreased. If the required densities are not being obtained, therefore, it is often expedient to change to a thinner lift to insure obtaining the specified density. The sheepsfoot roller can be used for all materials except possibly cohesionless, granular materials. If the roller tends to "walk out" before proper compaction is obtained, the soil may be scarified lightly behind the roller during the first two or three passes to prevent this occurrence, and additional weight added to the roller. In a soil that has the proper moisture content and lift thickness, foot contact pressure and number of passes are the important variables affecting the degree of compaction obtained by sheepsfoot rollers. Most available sheepsfoot rollers are equipped with feet having contact area of from 5 to 8 square inches. The foot pressure can be varied by varying the amount of ballast in the drum or, in special cases, by welding plates of larger area on the faces of the feet. For the most efficient operation of the roller, the contact pressure should be close to the maximum at which the roller will "walk out" satisfactorily (within about 2 inches of the surface) at about the number of passes necessary to produce the required density. The desirable foot-contact pressure varies for different soils, depending on the bearing capacity of the soil, and therefore the proper adjustments have to be made in the field based on observation of the roller. If the feet of the roller tend to "walk out" too quickly (for example, after two passes) then bridging may occur, and the bottom of the lift may not get sufficient compaction. This would

indicate that the roller is too light or the feet too large. If, on the other hand, the roller shows no tendency to "walk out" within the required number of passes, then the indications are that the roller is too heavy and the pressure on the roller feet is exceeding the shearing strength of the soil. After making the proper adjustments in foot pressure (by changing roller weight or, in special cases, foot size), then the only other variable is the number of passes. For economy, the number of passes necessary with the roller to obtain the required density (especially where a large number of passes are required) shall be compared with the number of passes required by other types of compaction equipment to obtain the identical compaction. The equipment requiring the smallest number of passes should be used.

j. Light Pneumatic Roller. As used in this manual, the term light pneumatic roller applies to the small rubber-tired roller, usually a "wobble wheel." The pneumatic roller is suitable for granular materials but is not recommended for fine-grained clay soils except as necessary for sealing the surface after a sheepsfoot roller has "walked out." Since it compacts from the top down, it will not compact a layer more than about 2 inches in thickness. It is used for finishing all types of materials, following immediately behind the blade and water truck.

k. Smooth-Wheel (Steel) Roller. Smooth-wheel rollers may be used in the compaction of soils, although that is not the purpose for which they are best suited. Both tandem and three-wheel types are used in the final rolling of subgrades and the rolling of base courses. If a smooth-wheel roller is used in earthwork the three-wheel type is generally preferred, because of the greater pressures exerted by the driving (rear) wheels. When three-wheel rollers are used, the soil generally should be spread in relatively thin layers, up to about 6 inches loose thickness in the usual case. With the heavier rollers with total weights of 10 to 12 tons, thicker lifts may sometimes be rolled satisfactorily, particularly in friable fine-grained soils. They are not effective on clean cohesionless sands, but may be used in compacting gravelly and clayey soils. In compacting clayey soils, care must be taken to roll a thin enough layer of soil so that compaction will be to the full depth, otherwise compaction may be limited to a surface crust.

l. Vibratory Rollers and Plates. Impact-type equipment such as compressed air or gasoline-driven hand tampers are effective in compacting soils and bases in small restricted areas. Vibratory-type compactors, roller or plate types, are also available for use in small restricted areas as well as for larger surface areas. Tests of vibratory compaction equipment indicate that it produces very high densities in cohesionless soils, but tends to bridge over in

cohesive soils. A few heavy pneumatic-tired rollers are also equipped or combined with vibrators, but available data indicates that the compacting capability of these rollers when the vibrator is operating is not appreciably greater than when the vibrator is not operating.

m. Equipment List. Column (12) and note 5, figure 61, list equipment that will usually produce the desired density with a reasonable number of passes when moisture conditions (par. 46) and lift thickness are properly controlled. See TM 5–252 for detailed descriptions of subgrade-construction equipment.

46. Moisture Control

a. Moisture Tests. Tests to determine the actual moisture content of a soil are not difficult to make (TM 5–530), but under normal procedures, samples are oven dried for 24 hours to obtain a dry weight. This limits severely the use of normal moisture determinations in moisture control. There are several methods of making "quick" moisture tests. Large, forced draft ovens can be used in connection with large diameter, flat samples. This method reduces the oven-drying time to about 2 hours, which speeds up the normal procedures. The sample may also be placed in a frying pan and dried over a hotplate or a field stove. The temperature is difficult to control in this procedure and organic materials may be burned, thus causing a slight to moderate error in the results. On large-scale projects where many samples are involved, the quick method may be used to speed up determinations by comparing the results obtained by this method with comparable results by oven-drying. Another quick method which may be useful is to put the damp soil in a metal cup with enough denatured grain alcohol to just cover, ignite the alcohol, and permit it to burn off. The alcohol method, if carefully done, will produce results equivalent to those obtained by careful laboratory drying except with plastic soils. For best results, it is suggested that the process of covering the soil with alcohol and burning it off completely be repeated three times. No attempt should be made to mix the alcohol with the soil.

b. Determination by Observation and "Feel." The number of tests that can be made even by the quick procedures is limited. The success of the moisture control in the field will depend principally on the ability of the engineer and construction foreman to judge the moisture content of the soil by observation and "feel."

(1) *Observation.* Estimates of the moisture content can be made by observing the workability of the material, and in the case of the sheepsfoot roller, by the penetration of the feet. A slight weaving or movement of the soil under

Table XIII. Moisture Content—U.S. Gallons/Sq Yd or Station Food for 6-In Compacted Lift

Moisture content		Dry unit weight (Pounds/cubic foot)																				
%	Gallons	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130	135	140	145	150
1	Sq yd	0.27	.30	0.32	0.35	0.38	0.41	0.43	0.46	0.49	0.51	0.54	0.57	0.59	0.62	0.65	0.68	0.70	0.73	0.76	0.78	0.81
	Station foot	3.0	3.3	3.6	3.9	4.2	4.5	4.8	5.1	5.4	5.7	6.0	6.3	6.6	6.9	7.2	7.5	7.8	8.1	8.4	8.7	9.0
2	Sq yd	0.54	0.60	0.65	0.70	0.76	0.81	0.86	0.92	0.97	1.03	1.08	1.14	1.18	1.24	1.30	1.36	1.40	1.46	1.51	1.57	1.62
	Station foot	6.0	6.7	7.2	7.8	8.4	9.0	9.6	10.2	10.8	11.4	12.0	12.7	13.1	13.8	14.4	15.1	15.6	16.2	16.8	17.4	18.0
3	Sq yd	0.81	0.89	0.97	1.05	1.13	1.22	1.30	1.38	1.46	1.54	1.62	1.70	1.78	1.86	1.94	2.03	2.11	2.19	2.26	2.35	2.43
	Station foot	9.0	9.9	10.8	11.7	12.6	13.6	14.4	15.3	16.2	17.1	18.0	18.9	19.8	20.7	21.6	22.6	23.4	24.3	25.1	26.1	27.0
4	Sq yd	1.08	1.19	1.30	1.40	1.51	1.62	1.73	1.84	1.94	2.05	2.16	2.27	2.38	2.48	2.59	2.70	2.81	2.92	3.02	3.13	3.24
	Station foot	12.0	13.2	14.4	15.6	16.8	18.0	19.2	20.4	21.6	22.8	24.0	25.2	26.4	27.6	28.8	30.0	31.2	32.4	33.6	34.8	36.0
5	Sq yd	1.35	1.49	1.62	1.76	1.89	2.03	2.16	2.30	2.43	2.57	2.70	2.84	2.97	3.11	3.24	3.38	3.51	3.65	3.78	3.92	4.05
	Station foot	15.0	16.6	18.0	19.5	21.0	22.5	24.0	25.5	27.0	28.5	30.0	31.6	33.0	34.6	36.0	37.6	39.0	40.6	42.0	43.6	45.0
6	Sq yd	1.62	1.78	1.94	2.11	2.27	2.43	2.59	2.75	2.29	3.08	3.24	3.40	3.56	3.73	3.89	4.05	4.21	4.37	4.54	4.70	4.86
	Station foot	18.0	19.8	21.6	23.4	25.2	27.0	28.8	30.6	32.4	34.2	36.0	37.8	39.6	41.4	43.2	45.0	46.8	48.6	50.4	52.2	54.0
7	Sq yd	1.89	2.08	2.27	2.46	2.65	2.84	3.02	3.21	3.40	3.59	3.78	3.97	4.16	4.35	4.54	4.73	4.91	5.10	5.29	5.48	5.67
	Station foot	21.0	23.1	25.2	27.3	29.4	31.5	33.6	35.7	37.8	39.9	42.0	44.1	46.2	48.3	50.4	52.5	54.6	56.7	58.8	60.9	63.0
8	Sq yd	2.16	2.38	2.59	2.81	3.02	3.24	3.46	3.67	3.89	4.10	4.32	4.54	4.75	4.97	5.18	5.40	5.62	5.83	6.05	6.26	6.48
	Station foot	24.0	26.4	28.8	31.2	33.6	36.0	38.4	40.8	43.2	45.6	48.0	50.4	52.8	55.2	57.6	60.0	62.4	64.8	67.2	69.5	72.0

9	Sq yd	2.43	2.67	2.92	3.16	3.40	3.65	3.89	4.13	4.37	4.62	4.86	5.10	5.35	5.59	5.83	6.08	6.32	5.56	6.80	7.05	7.29
	Station foot	27.0	29.7	32.4	35.1	37.8	40.5	43.2	45.9	48.6	51.3	54.0	56.7	59.4	62.1	64.8	67.5	70.2	72.9	75.5	78.3	81.0
10	Sq yd	2.70	2.97	3.24	3.51	3.78	4.05	4.32	4.59	4.86	5.13	5.40	5.67	5.94	6.21	6.48	6.75	7.02	7.29	7.56	7.83	8.10
	Station foot	30.0	33.0	36.0	39.0	42.0	45.0	48.0	51.0	54.0	57.0	60.0	63.0	66.0	69.0	72.0	75.0	78.0	81.0	84.0	87.0	90.0

the roller is evidence that the soil is just barely above optimum moisture content. A large amount of weaving is evidence of an excessive amount of moisture.

(2) *"Feel."* The engineer can develop ability to "feel" the soil between his fingers and estimate the moisture. This ability can be developed by rubbing soils between the fingers at various moisture contents, making a record of the feel, and comparing the notes with the actual moisture contents. Special samples can be prepared by the project laboratory at definite moisture contents to assist the engineer in developing his ability.

c. Quantitative Moisture Control.

(1) If the moisture content of the soil is less than optimum, the amount of water which must be added for efficient compaction generally is computed in gallons per 100 feet of length (station). The computation is based upon the dry weight of soil contained in a compacted layer. For example, assume that the soil is to be placed in a layer 6 inches in compacted thickness at a dry unit weight of 120 pounds per cubic foot. The moisture content of the soil is determined to be 5 percent, while the optimum moisture content is 12 percent. Assume that the strip to be compacted is 40 feet wide or 40 station feet per station. The weight of water required per cubic foot (with *no* allowance for evaporation) is equal to $120\ (0.12 - 0.05) = 120(0.07) = 8.4$ pounds. Allowance for evaporation varies from 5 percent to 10 percent depending on temperature, humidity, work conditions, type of soil, and length of time between placing and rolling. In this example an allowance of 10 percent is made for evaporation and consequently, the 8.4 is divided by $1.00 - .10 = 0.90$. The weight of water per cubic foot of soil then is $8.4/0.90 = 9.33$ pounds. Since water weighs 8.34 pounds per gallon, the number of gallons per cubic foot of soil $= 9.33/8.34 = 1.119$. The volume of compacted soil per station $= 40\ (100)\ 6/12 = 2{,}000$ cubic feet. The amount of water required per station to insure compaction at optimum moisture content then is $2{,}000\ (1.119) = 2{,}238$ gallons.

(2) Computations of the type discussed in (1) above can be avoided (except for evaporation allowance) by the use of table XIII. Tables XIII and XIV are taken from an article by Lt. Col. A. C. Einbeck entitled "Tables for Moisture Control," Volume 52, Nr. 346, *The Military Engineer*, March–April 1960.

(3) Adjustment for the effect of rain during the laydown may be made by use of table XIV.

Table XIV. Effect of Rainfall

Inches of rain	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Gals/sq yd.	0.56	1.12	1.68	2.24	2.80	3.36	3.93	4.49	5.05	5.61
Gals/station foot.	6.22	12.4	18.7	24.9	31.1	37.3	43.7	49.9	56.1	62.3

(4) The following example will show how the tables may be used:

Base Data—for station 10 + 00 to 25 + 00

Number of stations	15
Average width of fill	100 feet
Area	1,500 station feet
Average depth to fill	3 feet
Dry unit weight of earth (Modified AASHO max density)	120 pounds/cubic foot
Initial moisture content	8%
Optimum moisture content	11%
Capacity of distributors	2,000 gallons

Solution—

Moisture to be added 11% − 8% = 3%	
From table XIII (3% for dry unit wt 120-lb)	= 21.6 gal/sta ft
Water required/6-in lift 1,500 × 21.6	= 32,400 gal
Distributor loads 32,400/2,000	= 16.2
Total water required between stations 10 + 00 to 25 + 00 32,400 × 36/6 (3-foot fill requires six 6-inch lifts)	= 194,400 gal
Distributor loads 194,400/2,000	= 97.2
Rate of application per lift from table XIII	= 1.94 gals/yd²
Number of passes required at 0.5 gal/yd² 1.94/0.50	= 3.88 say 4/lift
4 × 36/6	= 24 for entire job

Base Data Continued—Between lifts on the fill, 0.25 inch of rain is recorded.

Solution—

From table XIV

$0.20''$ rain = 1.12 gal/yd^2 or 12.4 gal/ sta ft

$0.05''$ rain = 0.28 gal/yd^2 or 3.1 gal/sta ft

total = 1.40 gal/yd^2 or 15.5 gal/sta ft

Total adjustment/lift = $15.5 \times 1{,}500 = 23{,}250$ gal

By inspection, three passes with the distributor are saved.

(5) From 5 to 10 percent allowance for evaporation should be added to values obtained from table XIII.

47. Compaction Requirements

a. Basic Considerations. For a given wheel load and depth below the surface a definite degree of subgrade and base-course compaction is needed to prevent detrimental settlement under traffic. Requirements stated in this paragraph depend upon whether a road or an airfield is being constructed; if an airfield, they are also dependent upon the class of airfield. Requirements stated herein reflect experience with equipment normally available and the use of proper construction procedures. In some situations, stated density requirements may not be practically attainable with available equipment; in such cases, compaction requirements must be established for the particular project involved. The theory of compaction is discussed in detail in TM '–541.

b. Roads. For hasty unsurfaced roads, somewhat less than 90 ›ercent of maximum modified AASHO density in subgrade and base is adequate. For more deliberate road construction, the subgrade should be compacted to not less than 90 percent of modified AASHO density. Base courses and cohesionless-sand subgrades should be compacted to not less than 95 percent of modified AASHO density.

c. Airfields. Definite degrees of compaction in the subgrade and base course must be specified in designing airfield pavements. The required compaction will prevent excessive loss of grade and development of small depressions in pavement known as bird baths. The assignment of definite degrees of compaction is also necessary because the design CBR values are based on assumed degrees of compaction. For full operational and minimum operational airfields, as a safety factor, *the design CBR will be based on 95 percent compaction as a maximum.* When designing a full operational field for B–58, B–47, B–52 aircraft the compaction requirements are shown on compaction curves, figures 77, 78, and

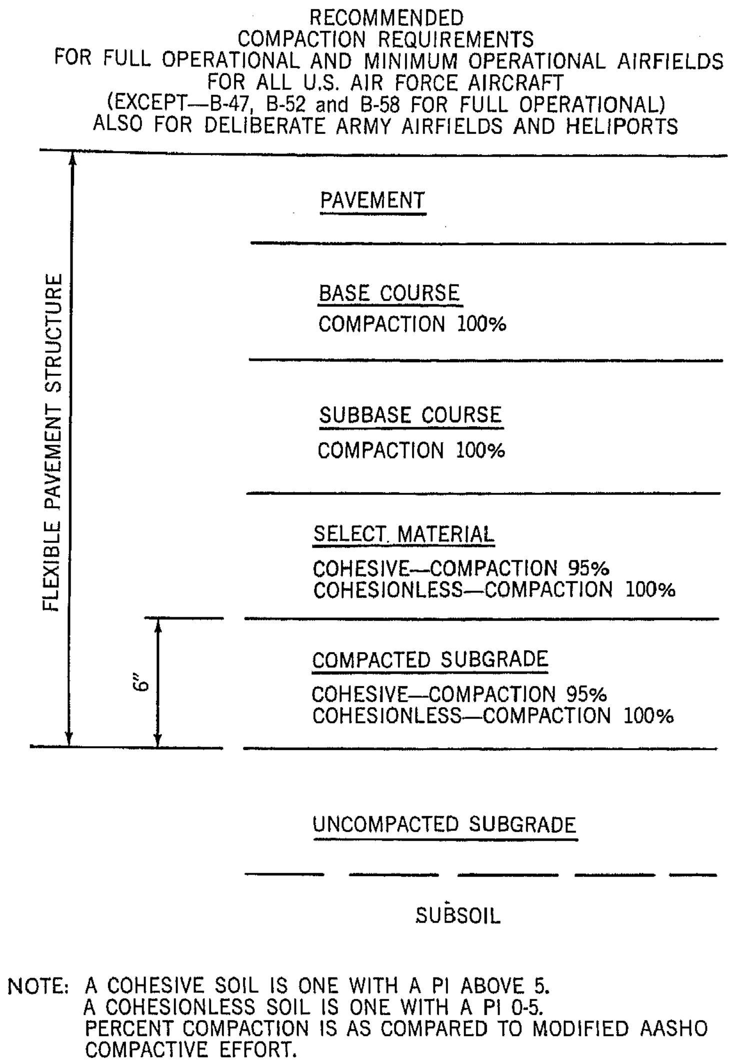


Figure 76. Recommended compaction requirements.

79 respectively. *For all lesser operational categories for above aircraft and for all other aircraft the compaction requirements shall be as shown on figure 76.*

d. Rigid Pavements Over Fill Sections. In cohesionless soils the top 6 inches of fills will be compacted to 100 percent of maximum modified AASHO dry unit weight; remaining depth to 95 percent. Cohesive soils in fill sections will be compacted to not less than 95 percent of modified AASHO dry unit weight.

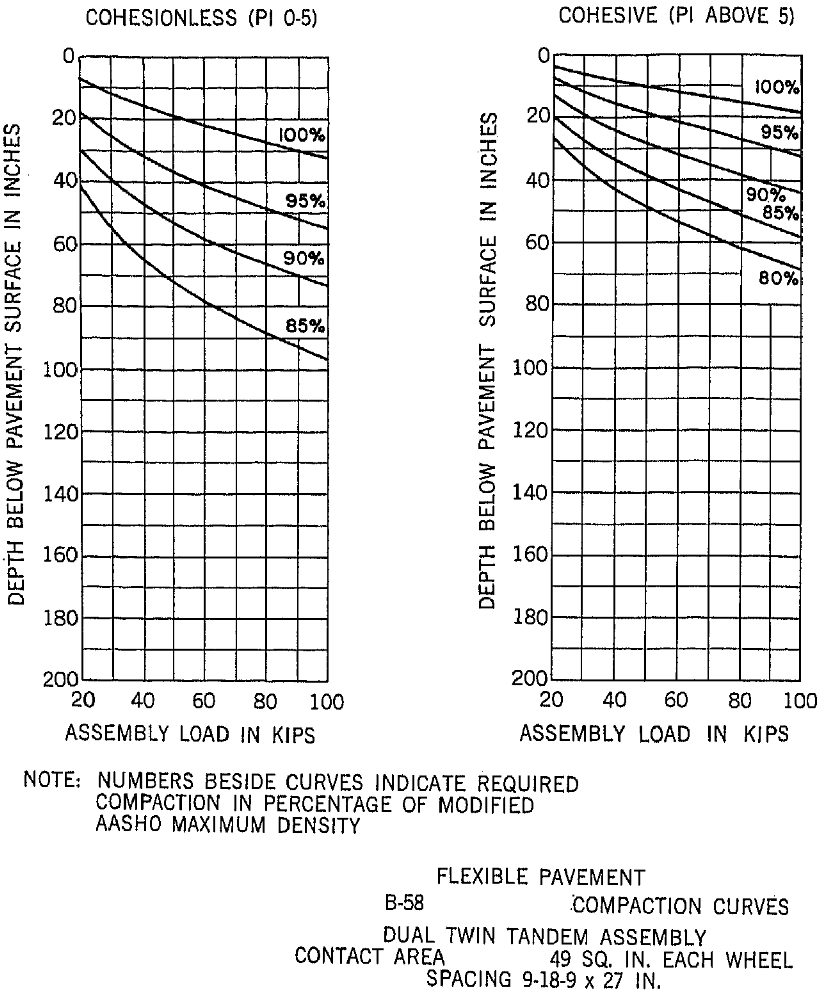
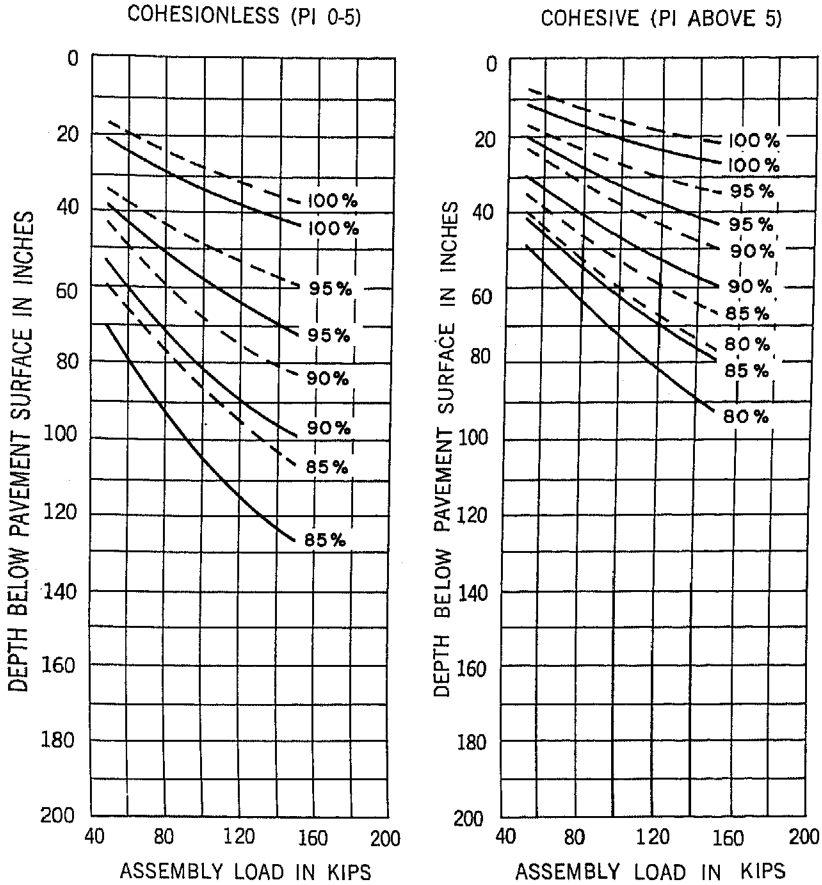


Figure 77. Compaction curves for B–58 aircraft.

e. Rigid Pavements Over Cut Sections. In cohesionless soils the top 6 inches will be compacted to 100 percent of modified AASHO dry unit weight; following 18 inches to 95 percent. Upper 6 inches of cohesive soil in cuts must be compacted to not less than 90 percent of modified AASHO dry unit weight.

48. Compaction Procedures: Normal Soils

a. Soil Conditions. The moisture content of the fill material is seldom in the specified range for compaction and the soil usually must be either wetted or dried before it is ready for compaction. Water cannot be added nor drying accomplished satisfactorily unless the material is pulverized. Material as excavated from cuts



NOTE: NUMBERS BESIDE CURVES INDICATE REQUIRED COMPACTION IN PERCENTAGE OF MODIFIED AASHO MAXIMUM DENSITY

FLEXIBLE PAVEMENT
B-47 COMPACTION CURVES
TWIN (BICYCLE) ASSEMBLY
CONTACT AREA 267 SQ. IN. EACH WHEEL
SPACING 37 IN.

——— TRAFFIC AREA B

– – – – TRAFFIC AREA C

Figure 78. Compaction curves for B–47 aircraft.

and borrow areas is not always in condition for adding water or drying. The following conditions may be encountered; methods for correcting the conditions are also given.

(1) Material that comes from the pit in a "chunky" condition and is below optimum moisture content:

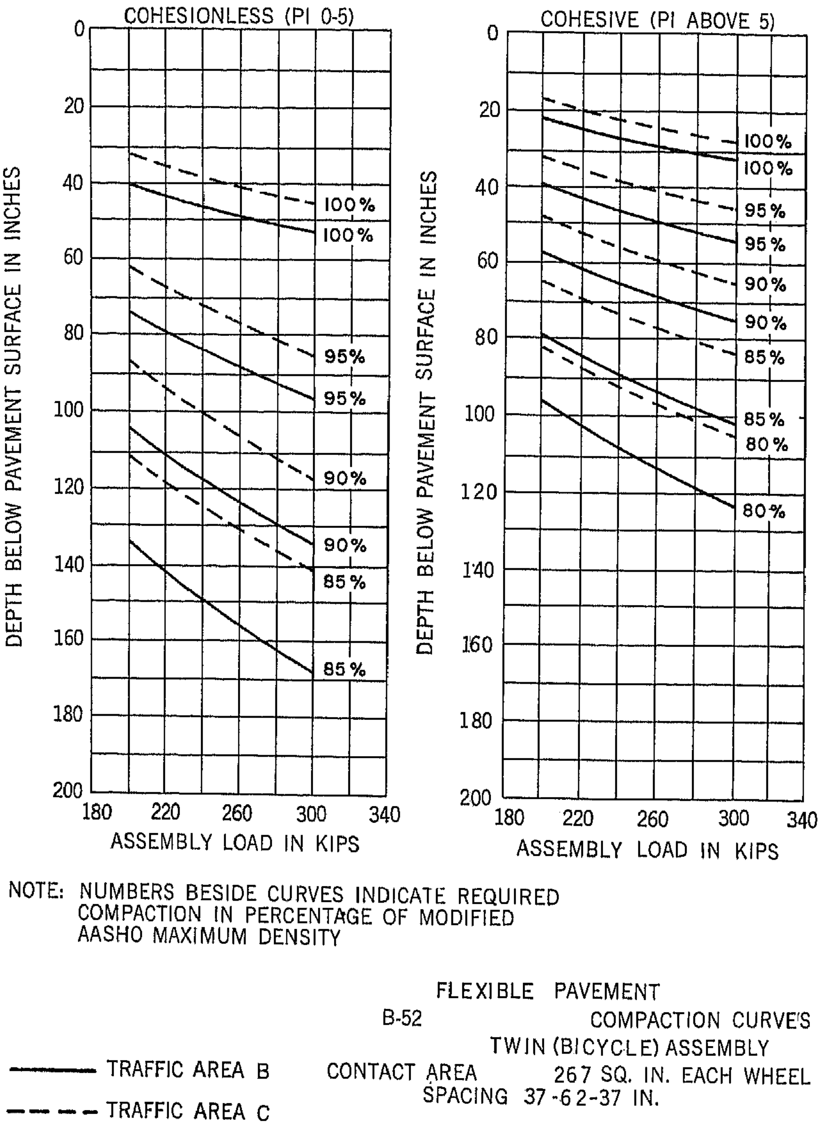


Figure 79. Compaction curves for B–52 aircraft.

(*a*) Blading and rolling with sheepsfoot roller will break down the chunks. Light applications of water may help.

(*b*) Blading and disking will break down the chunks. Light applications of water may help.

(*c*) Working the material with a pulvimixer, plus light applications of water will break down the chunks.

(*d*) "Chunk" material of low plasticity will break down under loaded equipment and/or "cat" tracks.

(2) Material that comes from the borrow pit in chunks and is above optimum moisture content:

(*a*) Specially designed equipment (pulvimixers) will mix and pulverize this material. Some aeration occurs during the process.

(*b*) Continuous blading and disking will pulverize the material and produce some aeration.

b. Fill. During the process of grading and filling, the subgrade is brought to the intended lines and grades indicated by grade stakes. In fill areas the subgrade soil is compacted by placing and spreading thin lifts and rolling with the proper equipment (fig. 61). The lift thickness should not exceed 6 inches after compaction where compaction to 90 percent or more of modified AASHO density is required. For hasty construction where less density is required, thicker lifts can be used—up to 9 inches for sheepsfoot rollers and up to 18 inches for the 50-ton pneumatic-tired roller. Soils should be compacted at a moisture content in the optimum range as established by the laboratory compaction procedure. It is recognized that moisture cannot be rigidly controlled in military construction, but certain practical limits must not be exceeded. Generally, plastic soils cannot be compacted when the moisture content is more than about one-fourth above or below the optimum percentage. Much better results will be obtained if the moisture content can be controlled to within one-tenth more or less than the optimum percentage. The moisture content of cohesionless soils is not as important, but some cohesionless sands tend to bulk at low moisture contents, and compaction should not be attempted until the moisture content is corrected. In general, they should be compacted with dozers or other tracked equipment or with rubber-tired or steel rollers and at a moisture content sufficiently high to prevent bulking.

c. Cuts. In most theater-of-operations construction, the soil in cut sections should be scarified, mixed in the rotary tiller if necessary, and rolled, but if the natural subgrade exhibits densities equal to the required modified AASHO density, scarifying and/or mixing is not required and no rolling is necessary except to provide a smooth surface. However, where pavements of the flexible type are constructed for the heavier wheel loads at more permanent installations, the subgrade compaction for soils that gain strength with compaction should meet the requirements given in paragraph 47. This may necessitate removing the soil and replacing and compacting it in successive lifts, in order to obtain the required densities at the greater depths.

49. Compaction Procedures: Special Soils

a. Clays That Lose Strength When Remolded. Certain clays, generally in the CH and OH group, will lose strength when remolded. These are clays which have been consolidated to a very high degree either under an overburden load or by alternate cycles of wetting and drying. They have a high strength in the undisturbed state, but scarifying, reworking, and compacting these soils in cut areas to the design density may produce a lower bearing value than that of the soils in place. When such clay soils are encountered in cuts, bearing values should be obtained for both the undisturbed soil and for the soil when remolded and compacted to the design density at the design moisture content. This procedure is generally applicable only to deliberate projects. If the undisturbed value is the higher, no compaction should be attempted and construction operations should be conducted to produce the least possible disturbance of the soil. Pavement design should be based on the bearing value of the undisturbed material.

b. Silts That Become "Quick" When Remolded. Experience has shown that some silts, silty sands, and very fine sands (predominantly ML and SM soils), when compacted in the presence of a high water table, will pump water to the surface and become "quaky" or "spongy" with the loss of practically all bearing value. These soils cannot be compacted unless they are dried. Their bearing value is good if they can be compacted at the proper moisture content, and every effort should be made to lower the water table so compaction can be accomplished. If the trouble occurs in local areas, such soils can be removed and replaced with less susceptible material. Where removal or drainage and subsequent drying cannot be accomplished, these soils should not be disturbed by attempting to compact them; instead, they should be left in the natural state, and additional thickness of base should be used to insure that the subgrade will not be overstressed.

c. Treatment of Soils With Expansive Characteristics. Soils with expansive characteristics occur and give the most trouble in certain areas of the West Gulf Coastal Plains and the Great Plains where climatic conditions are conducive to significant changes in moisture content of the subgrade during different seasons of the year. Such soils can also cause trouble in any region where construction is accomplished in a dry season and the soils absorb moisture during a subsequent wet season. If highly compacted, these soils will swell and produce uplift pressures of considerable intensity if the moisture content of the soil increases after compaction. This action may result in differential heaving of flexible pavements that is intolerable. A common

method of treating a subgrade with expansive characteristics is to compact it at a moisture content and to a unit weight which will minimize expansion. The proper moisure content and unit weight for compaction control of a soil with marked expansion characteristics are not necessarily the optimum moisture content and unit weight determined by the modified AASHO compaction test, but may be determined from a study of the relationship between moisture content, unit weight, percent swell, and CBR for a given soil. Generally, the minimum amount of swell and the highest soaked CBR will occur at a molding moisture content slightly wet of optimum. The required moisture content and unit weight may be determined as part of the CBR test or by separate expansion tests. Field control of the moisture content must be carefully exercised since, if the soil is compacted too dry, the expansion will increase; and if compacted too wet, low unit weight will be obtained and the soil will shrink during a dry period and then expand during a wet period. This method requires detailed testing and extensive field control of compaction; hence, in some cases it may be desirable to construct a base of sufficient thickness to insure against harmful effects of expansion. Compaction requirements may govern the design in these areas.

50. Blanket Courses

Wherever the subgrade consists of plastic and cohesive soil, and the base course to be laid upon it consists of crushed rock or coarse gravel, a 1- to 2-inch layer of sand or screenings should be spread upon the subgrade and rolled to the maximum possible density. Such a blanket course helps prevent the plastic soil from working up into the coarse base material under traffic during wet conditions. Experience has shown that if the criteria given in paragraph 12*d*(1) for filter design are followed, no movement of the fines will be experienced. As discussed previously in paragraph 12*d*, a good criterion for two contiguous layers is that the 15 percent size of the coarser layer shall not be greater than 5 times the 85 percent size of the finer layer. This criterion should be followed for all layers of soil in contact with one another, regardless of whether they are blanket layers or base course and subgrade layers. Sprinkling during rolling is desirable. A blanket course is desirable but not necessary if the base material is dense-graded.

CHAPTER 7

SOIL STABILIZATION

Section I. Introduction

51. Basic Considerations

a. Definition. Soil stabilization is a method of processing soil in order to render it more suitable for the use for which it is intended under the prevailing traffic and climatic conditions. In other words, the objective of soil stabilization, as related to roads and airfields, is to produce a firm soil mass which is capable of withstanding the stresses imposed upon it by traffic loads in all kinds of weather without excessive deformation. The term is further generally applied to the processing of a layer of soil of predetermined thickness which is to serve as a subgrade, base course, or wearing surface. The construction of bases or wearing surfaces from inherently stable materials, such as crushed rock, is not generally regarded as soil stabilization.

b. Purpose. Soil stabilization is most frequently used to improve the existing subgrade soil so that its bearing capacity is increased sufficiently that the thickness of the base course may be reduced, or to eliminate the need for a separate base course of imported materials. A stabilized soil mixture is often used to provide a base course for the support of a relatively thin bituminous wearing surface.

52. Importance of Drainage and Compaction

The importance of adequate drainage and compaction in contributing to soil stabilization cannot be overemphasized. Many subgrade soils which, when first examined, appear to be unsatisfactory, may be made to fulfill their intended function satisfactorily by the provision of adequate drainage facilities and proper compaction. Other soil-stabilization procedures are frequently expensive and time consuming. They should not be undertaken unless it is certain that drainage and compaction alone cannot accomplish the intended objective. Similarly, adequate drainage and proper compaction are essential to the proper construction and functioning of soils stabilized by other means, when it is determined that such soil stabilization is necessary.

53. Types of Stabilization

Several types of soil stabilization are of consequence in military construction. By far the most important of these is *mechanical*

stabilization, which includes compaction alone as well as the blending of two or more soil materials in order to produce a soil mixture which has the desired properties. Commercial materials are also used in soil stabilization, when soil materials needed for blending with the existing subgrade soil are not easily or economically available in the area. The most important commercial materials which may be available to the military engineer are bituminous materials (asphalt or tar), portland cement, and lime. Soil stabilization using bituminous materials is termed *bituminous soil stabilization* (pars. 63–66). When cement is used as a stabilizing agent, the mixture produced is called *soil cement* (pars. 67–69). A number of chemicals have been used for stabilization in the field or laboratory. Most of these are not important to the military engineer from a practical standpoint, since most chemicals are not normally available in field situations. Perhaps the most important of these chemical materials, in the sense that it is frequently available in the field, is lime, which is useful in the treatment of certain cohesive soils. Lime soil stabilization is discussed briefly in paragraph 70.

Section II. MECHANICAL STABILIZATION

54. Essential Requirements

The three essentials for obtaining a properly stabilized soil mixture are proper gradation, a satisfactory binder soil, and proper control of the moisture content. To obtain uniform bearing capacity, uniform mixing and blending of all materials is essential. The mixture must be compacted at or near optimum moisture content (par. 46) in order to obtain satisfactory densities and strengths.

a. The primary function of the portion of a mechanically stabilized soil mixture which is retained on a No. 200 sieve is to contribute internal friction. Practically all materials of a granular nature which do not soften when wet or pulverize under traffic can be used, although the best aggregates are those which are made up of hard, durable, angular particles. The gradation of this portion of the mixture is important, because the most suitable aggregates, generally speaking, are those which are well graded from coarse to fine. Well-graded mixtures are preferred because they generally have greater stability when compacted than do poorly graded mixtures, can be compacted more easily, and have greater increases in stability with corresponding increases in density. Materials which are satisfactory for this use include crushed stone, crushed and uncrushed gravel, sand, and crushed slag. Many other locally available materials have been successfully

used, including disintegrated granite, talus rock, mine tailing, caliche, coral, limerock, tuff, shell, clinkers, cinders, and iron ore. When local materials are used, requirements relative to proper gradation cannot always be met. If conditions are encountered in which the gradations obtained by blending local materials are such as to be either finer or coarser than the specified gradation, it is desirable to satisfy the size requirements of the finer fractions and neglect the gradation of the coarser sizes.

b. The portion of the soil which passes a No. 200 sieve functions as a filler for the rest of the mixture and supplies cohesion, which will aid in the retention of stability during dry weather. Clay or other plastic soils or highly organic soils will not be used to stabilize sand subgrade. The nature and amount of this finer material must be carefully controlled, since an excessive amount of it will result in an excessive change in volume with change in moisture content and the development of other undesirable properties.

c. The properties of the soil binder usually are controlled by controlling the plasticity characteristics, as indicated by the liquid limit and plastically index. These tests are performed on the portion of the material which passes a No. 40 sieve. The amount of fines is controlled by limiting the amount of material which will pass a No. 200 sieve. Normally, material passing a No. 200 sieve shall be limited to a maximum of 15 percent. When the stabilized soil is to be subjected to frost action, this factor must be kept in mind when designing the soil mixture.

55. Objectives and Limitations of Soil Blending

a. The overall objective of soil blending is simply to blend available soils in such a fashion that, when properly compacted, they will give the desired stability. In one sense, the goal is to achieve a soil combination which is as near the top of figure 59 as possible. In certain areas, for example, the natural soil at a selected location may be of low supporting power because of an excess of clay, silt, or fine sand. Within a reasonable distance there may exist suitable granular materials which may be blended with the existing soil to effect a marked improvement in stability of the soil at a very much lower cost in manpower and materials than is involved in applying imported surfacing, and may even produce better results.

b. Without minimizing the importance of soil blending, limitations of this method should also be realized. The principles of mechanical stabilization have frequently been misused, particularly in areas where frost action is a factor in the design. For example, clay has been added to granular subgrade materials

in order to "stabilize" them, when in reality all that was needed was adequate compaction to provide a strong, easily-drained base which would not be susceptible to detrimental frost action. An understanding of the compaction which can be achieved by modern rolling equipment should prevent a mistake of this sort. Somewhat similarly, poor trafficability of a soil during construction because of a lack of fines should not necessarily provide an excuse for mixing in clay binder. It may be possible to solve the problem by applying a thin surface treatment or some other expedient method.

56. Requirements

a. Gradation requirements for mechanically stabilized soil mixtures which are to serve as base courses are given in table XV. Experience in civil highway construction has indicated that best results will be obtained with this type of mixture if the weight of the fraction passing the No. 200 sieve is not greater than one-half of the fraction passing the No. 40 sieve. The size of the largest particles should not exceed one-half of the thickness of the layer in which they are incorporated. The mixture should be well graded from coarse to fine.

Table XV. Desirable Gradation for Crushed Rock, Gravel, or Slag, and Uncrushed Sandy and Gravel Aggregates for Base Courses and for Mechanical Stabilization

Sieve designation	Percent passing each sieve (square openings) by weight				
	Maximum aggregate size				
	3-inch	2-inch	1½-inch	1-inch	1-inch sand-clay
3-inch	100				
2-inch	65–100	100			
1½-inch	----------	70–100	100		
1-inch	45–75	55–85	75–100	100	100
¾-inch	----------	50–80	60–90	70–100	
⅜-inch	30–60	30–60	45–75	50–80	
No. 4	25–50	20–50	30–60	35–65	
No. 10	20–40	15–40	20–50	20–50	65–90
No. 40	10–25	5–25	10–30	15–30	33–70
No. 200	3–10	0–10	5–15	5–15	8–25

b. A basic requirement of soil mixtures which are to be used as base courses, unless higher requirements are specified, is that the plasticity index of each component of the mixture should not exceed 5, and the liquid limit should be less than 25. Under certain circumstances, this requirement may be relaxed if a satisfactory bearing ratio is developed. In theater-of-operations con-

struction, the requirements for each component may be lowered to a liquid limit of 36 and a plasticity index of 10 for full-operational airfields, and to a liquid limit of 45 and a plasticity index of 15 for emergency and minimum-operational airfields, when good drainage is provided. Where requirements are relaxed from the basic PI of 5 and LL or 25, severe maintenance problems can be expected under adverse weather or ground-water conditions.

c. If the base is to function satisfactorily, other requirements than those relating to the soil mixture alone must be met. The base must normally have a high bearing value. Density requirements and those relating to frost action are also of particular importance.

57. Rule-of-Thumb Proportioning

Satisfactory mixtures of the type described in the previous paragraph are difficult to design and build without laboratory control. A rough estimate of the proper proportions of available soils in the field is possible, and depends upon manual and visual inspection. Suppose that a loose sand is the existing subgrade soil, and it is desired to add another soil from a nearby borrow source to achieve a stabilized mixture. Each soil should be sprinkled to the point where it is moist, but not wet. In a wet soil the moisture can be seen as a shiny film on the surface. What is desired is a mixture which will feel gritty and in which the sand grains can be seen. Also, when the soils are combined in the proper proportion, a cast formed by squeezing the moist soil mixture in the hand will not be either too strong or too weak; it should just be able to withstand normal handling without breaking. Several trial mixtures should be made until this consistency is obtained, being careful to note the proportion of each of the two soils. If gravel is available, this may be added, although there is no real rule of thumb to tell how much should be added. It is always desirable to have too little gravel rather than too much.

58. Blending

a. It is assumed in this discussion that an existing subgrade soil is to be stabilized by the addition of a suitable borrow soil to produce a mixture which will meet the specified requirements. The mechanical analysis and limits of the existing soil will generally be available from the results of the subgrade soil survey (pars. 24–29); similar information is necessary concerning the borrow soil. The problem then is to determine the proportions of these two materials which should be used to produce a satisfactory mixture. In some cases more than two soils must be blended to produce a suitable mixture. This situation is to be avoided when

possible because of the difficulties frequently encountered in getting a uniform blend of more than two local materials.

b. Trial combinations are usually made on the basis of the mechanical analysis of the soils concerned. In other words, calculations are made to determine the gradation of the combined materials and the proportion of each component adjusted so that the gradation of the combination will fall within specified limits. The plasticity index of the selected combination is then determined and compared with the specification. If this value is satisfactory, then the blend may be assumed to be satisfactory, provided that the desired bearing value is attained. If the plasticity characteristics of the first combination are not within the specified limits, then additional trials must be made. The proportions finally selected, meeting both the gradation and plasticity specifications, may then be used in the field construction process.

59. Numerical Example of Proportioning

This process of proportioning will now be illustrated by a numerical example. Two materials are available, material B in the roadbed, and material A from a nearby borrow source. The mechanical analysis of each of these materials is given in *a* below, together with the liquid limit and plasticity index of each. The desired grading of the combination is also shown, together with the desired plasticity characteristics.

a. Results of Laboratory Tests.

Mechanical Analysis

Sieve designation	Percentage passing, by weight		
	Material A	Material B	Specified
1-inch	100	100	100
¾-inch	92	72	70–100
⅜-inch	83	45	50–80
No. 4	75	27	35–65
No. 10	67	15	20–50
No. 40	52	5	15–30
No. 200	33	1	5–15

Plasticity Characteristics

Liquid Limit	32	12	not over 28
Plasticity Index	9	0	not over 6

b. Proportioning to Meet Specified Gradation. Proportioning of trial combinations may be done either arithmetically or graphically. A graphical method is illustrated in figure 80. The

two dashed vertical lines on the chart represent the outer limits of combinations of the two materials which will meet the specified gradation requirements. The boundary on the left represents the combination of 44 percent material A and 56 percent material B; the position of this line is fixed by the upper limit of the requirement relating to the material passing the No. 200 sieve (15 percent). The boundary on the right represents the combination of 21 percent material A and 79 percent material B; this line is established by the lower limit of the requirement relative to the fraction passing the No. 40 sieve (15 percent). Any mixture falling within these limits will satisfy the grading requirements. For purposes of illustration, assume that a combination of 30 percent material A and 70 percent material B is selected for a trial mixture. A similar diagram can be prepared for any two soils. The proportioning can also be done arithmetically. Graphical methods for proportioning more than two soils are more complex and are not included here.

60. Field Proportioning

In the field, the materials used in a mechanically stabilized soil mixture probably will be proportioned by loose volume. Assume that a mixture incorporates 75 percent of the existing subgrade soil, while 25 percent will be brought in from a nearby borrow source. It is desired to construct a layer which has a compacted thickness of 6 inches. It is estimated that a loose thickness of 8 inches will be required to form the 6-inch compacted layer; a more exact relationship can be established in the field as construction proceeds. Of the 8-inch loose thickness, 75 percent, or 0.75 (8) = 6 inches, will be the existing soil. The remainder of the mix will be the borrow soil, which will be spread in a 2-inch loose layer on top of the existing soil. The two materials then will be mixed thoroughly to a depth of 8 inches, and compacted by rolling. The proportions may be more accurately controlled by weight, if weight measurements can be made under existing conditions.

61. Use of Local Materials

The essence of mechanical soil stabilization is the use of locally available materials. Desirable requirements for bases of this type have been given above. It is quite possible, especially under emergency conditions, that mixtures of local materials will give satisfactory service, even though they do not meet the stated requirements. Many stabilized mixtures have been made using shell, coral, soft limestone, cinders, marl, and other materials previously listed. Reliance must be placed upon experience, an under-

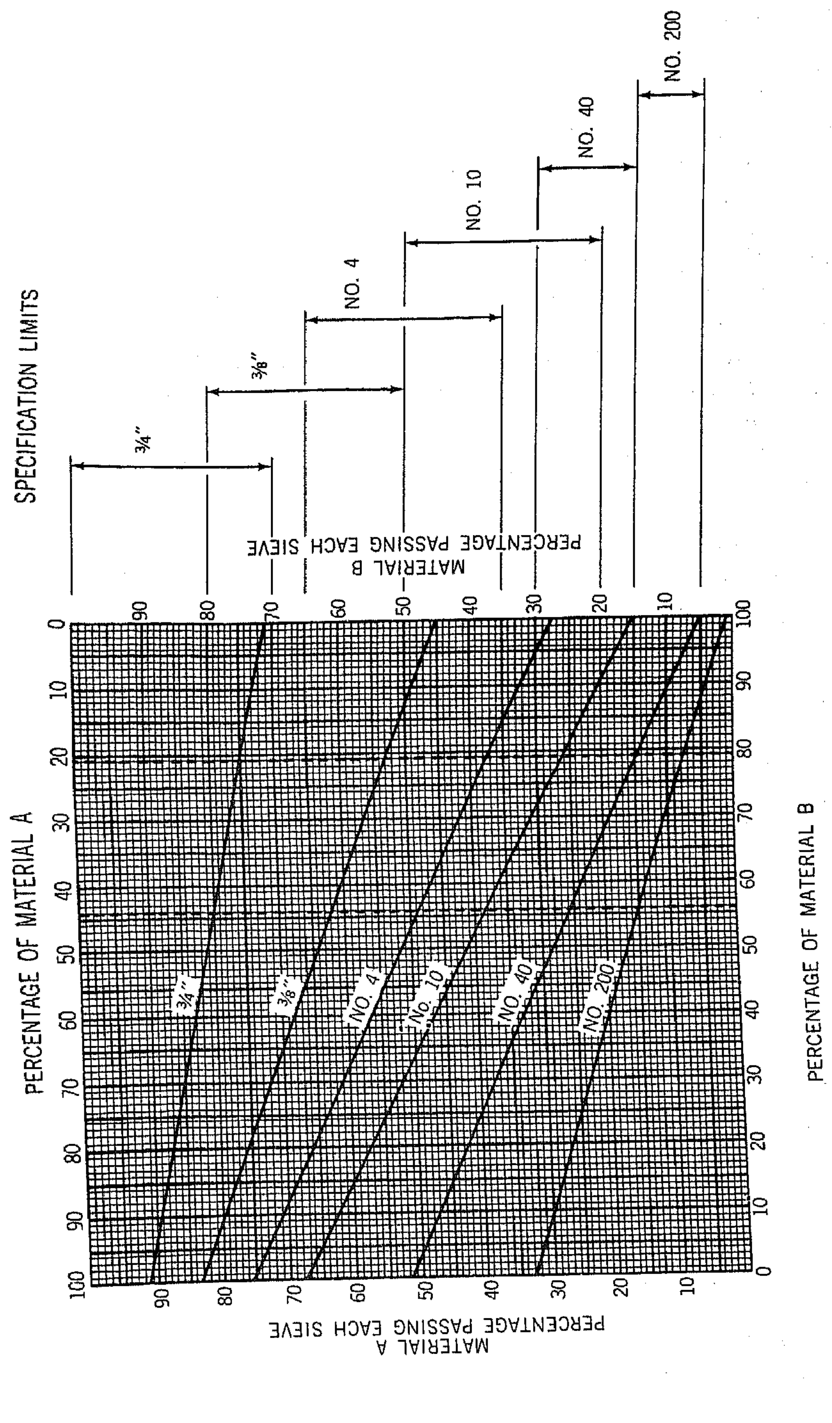


Figure 80. Graphical method of proportioning soils to meet gradation requirements.

standing of soil action, the qualities that are desired in the finished product, and other factors of local importance in proportioning such mixtures in the field.

62. Construction Procedures

a. The steps followed in the construction of a mechanically stabilized soil base or wearing surface will depend upon the equipment available and the soils. It is assumed in this discussion that the preliminary steps of cutting drainage ditches, shaping the road to the required crown and grade, removing organic materials and undesirable soils, and similar operations have been performed. Since subgrade soils are typically nonuniform, both in nature and extent, sufficient sampling must be carried out so that variations within the top few inches will be detected and accounted for in the design of the various sections.

b. Thorough pulverization and blending of the soil mixture is an absolute necessity for the success of this method. Tools used for this purpose include blade graders, harrows, disks, and rotary tillers (par. 45). When mixing is complete, the layer has a uniform color and appearance throughout the processed depth. If field conditions permit, frequent samples should be taken of the combined materials in order to detect any sections which deviate from the desired mixture. Some variation is to be expected in this type of construction and should be corrected by adding the necessary soil components before work is continued.

c. When the necessary equipment is available, mixing operations may be performed with a traveling plant or, occasionally, at a central mixing plant.

d. When the mixing operation is finished, the soil is compacted, using the type of equipment which is best suited to the soil involved. The soil should be compacted at or near optimum moisture and in accordance with the compaction requirements of paragraph 47. If the soil is too dry, water is added by sprinkling and thoroughly mixed with the soil, using the tools indicated in *b* above. If too wet, the soil must be dried until the moisture content reaches the desired amount. The best results can usually be obtained by compacting the initial lift of a stabilized base course by using pneumatic-tired rollers. This is particularly true where the underlying soil is not stable and a sheepsfoot would dissipate the stabilized base or change the gradation by mixing in unstabilized soil. In laying any stabilized base, a small windrow of material should be left to fill in sheepsfoot indentations, where a sheepsfoot roller is used for compaction, and to obtain a uniform surface. An alternative method is to use a light pneumatic-tired roller to compact the upper 1½ to 2 inches of disturbed material. Final

rolling should be accomplished by either a pneumatic roller or a steel wheel roller.

e. After compaction, the surface is shaped by light blading and rolled with three-wheel steel rollers, accompanied by sprinkling as necessary, until the surface is true to line and grade and has a tight, uniform surface texture.

Section III. BITUMINOUS STABILIZATION

63. Description

Bituminous soil stabilization refers to a process by which a controlled amount of bituminous material is thoroughly mixed with an existing soil or aggregate material to form a stable base or wearing surface. The bituminous material used may be either asphalt or tar, although asphalts are more frequently used. Asphalt emulsion or low-viscosity cutback asphalts which are liquid at normal temperatures and require no, or very little, heating at the time of application generally are preferred for this use. The bituminous material may be used for one, or both, of two general purposes. The first is to increase the resistance to deformation by supplying adhesion through the cementing action of the bituminous material. Second, in cases where exposure to water is a problem, a degree of waterproofing is achieved by the incorporation of the bituminous material, thus aiding in retention of a higher shearing strength than would otherwise be normal.

64. Uses and Conditions for Use

a. Uses. General uses of bituminous soil stabilized mixtures include the conversion of an otherwise unsatisfactory base-course material into a satisfactory one, the creation of a working floor on cohesionless-sand subgrades, and the production of low-cost wearing surfaces to carry light traffic.

b. Conditions for Use. Bituminous soil stabilization should not be considered for use except in situations where suitable base-course materials are not economically available, or where it is not feasible to mechanically stabilize the existing soil to produce the desired bearing value. The production of this type of mixture requires that the soil be thoroughly pulverized and mixed with the bituminous material. This requirement generally limits the economical use of this method to the treatment of essentially cohesionless soils, including sands, sandy and gravelly soils, and the more friable silts. Cohesive soils have been successfully stabilized with bitumen. However, because of the difficulties usually encountered in the pulverization and mixing processes, this practice is not recommended. For successful results, the moisture content of the soil must be reduced to fairly low values before the bituminous

material is incorporated, and the soil-bitumen mixture must be thoroughly cured to remove volatiles before compaction, especially if cutback asphalts are used. These requirements, in general, limit the use of this type of construction to periods of good weather.

65. Materials Used

a. Soil. As indicated above, the soils which are most suitable for stabilization with bituminous materials are those which can be easily pulverized and mixed with available equipment. Soils with high silt and clay content are not regarded as being suitable for this type of stabilization. Soils containing a large amount of mica are also not suitable for this type of construction. Friable soils that are inherently somewhat stable and those that can readily be improved by the addition of other soils are best suited for bituminous stabilization. Experience on civilian airfields in the United States indicates that soils for bituminous stabilization should not contain more than 30 percent of material passing a No. 200 sieve, their liquid limit should be less than 40, and their plasticity index less than 12.

b. Bituminous Materials. It is generally desirable to use bituminous materials which require the least heating in this work. Of the asphaltic products, rapid-curing cutback asphalts, grades RC–1 to RC–4, and medium-curing cutback asphalts, grades MC–1 to MC–4, are most frequently used. The heavier grades require considerable heating to use. See TM 5–337 for mixing and spraying temperatures. Grades RC–1 and RC–2 have been extensively used in the stabilization of cohesionless sands. Slow-setting asphalt emulsions are also used frequently and are sometimes used with moist soils. Slow-curing liquid asphalts or "road oils" have been used to some extent, particularly with soils which contain an excess of fines; their very long curing time makes them less useful than the cutback asphalts. Tars have been used, particularly grades RT–3 to RT–7, in areas where these materials are readily available; their use is not recommended in surface courses. The grade of bituminous material used will depend upon those available, the soil, and the mixing equipment. Lighter grades are generally preferred with soils which contain little or no fines, and for in-place mixing with commonly available tools.

c. Mixtures. Satisfactory bituminous soil-stabilized mixtures generally contain from 4 to 9 percent bitumen by weight exclusive of the volatile materials. The amount of bitumen generally increases with an increasing amount of fines in the soils. Although laboratory procedures have been developed for determining the proper amount of a given bituminous material to be used with a given soil, these methods have not been standardized and are

not generally available to the military engineer in the field. If a soils test set No. 12 is available, the coated aggregates can be heated in an oven to drive off volatiles and compacted in a mold. The modified Marshall stability tests (TM 5–337) run on a series of samples will give a strength indication as well as optimum asphalt content.

66. Construction Procedures

a. Preliminary steps in bituminous soil stabilization are generally the same as those described for mechanical soil stabilization (par. 62).

b. Before application of the bituminous material, the soil must be pulverized and mixed to a uniform condition to the desired depth of treatment. A general requirement is that the moisture content of the soil should be less than 2 percent, although some sands can be handled successfully at moisture contents slightly in excess of this. Otherwise, the water will adhere to the soil particles and the bitumen will not coat the material properly. When asphalt emulsion is used, the moisture content of the soil may be somewhat greater, depending on the soil involved. After the soil has been aerated and the moisture content reduced sufficiently, the bituminous material is applied by means of a pressure distributor, if mix-in-place methods are being used. A travel plant can also be used for mixing soil and cutback asphalt for soil stabilization.

c. The bituminous material must be thoroughly and uniformly mixed with the soil, using equipment such as plows, harrows, cultivators, blade graders, and rotary tillers (par. 45). This operation may be done in a traveling plant, as described in TM 5–252. It may also, on occasion, be done in a central mixing plant. Plant mixtures are generally more uniform than those produced by road mixing, and the operation is less affected by adverse weather conditions.

d. After mixing has been completed, aeration must be continued until virtually all of the volatile materials have been removed. This is absolutely essential in this type of construction in order for the mixture to cure and harden properly. The mix may then be compacted, using the type of equipment best suited to the operation. When cohesive soils are stabilized in this fashion, water is sometimes added to facilitate mixing and compaction, but the compacted mixture must still meet the requirements of *e* below. Finishing operations are similar to those described under mechanical stabilization.

e. If the mixture is to be used as a base course, an additional curing time may be necessary before the surface is sealed or a wearing surface constructed. The base should not be sealed or

surfaced until the moisture content has dropped to the range of 2 to 4 percent.

Section IV. SOIL-CEMENT AND SOIL-LIME STABILIZATION

67. Basic Considerations

a. Soil-cement is an intimate mixture of pulverized soil and portland cement. It is moistened, compacted, and cured during construction, and hardens into a semirigid material which possesses considerable compressive strength. The amount of cement used is usually from 7 to 14 percent by volume of the compacted mixture.

b. Soil-cement is principally used in the construction of base courses, since it cannot withstand successfully the abrasive effects of traffic and direct exposure to climatic changes over an extended period of time. Soil-cement may serve as a wearing surface under emergency conditions for a brief period of time. Perhaps the most frequent use of soil-cement mixtures is as base courses which are covered by relatively thin bituminous surface treatments. Construction with soil-cement is generalily limited to use as a base course directly under a bituminous wearing surface for pavements subject to gross loads of not more than 30,000 pounds. Soil-cement may also serve as a base for higher type pavements which will be subjected to heavy traffic.

c. As with bituminous soil stabilization, soil-cement should not be considered for military construction unless it is certain that materials are not economically available for mechanical stabilization. Practically all soils can be stabilized with cement, but not all soils can be stabilized easily and economically by this method. Thorough pulverization of the soil and thorough mixing with cement and water are prerequisites to success. Hence, for military construction, soil-cement stabilization is limited to granular and friable soils which can be pulverized readily. Soils which have a high silt or clay content require more cement and present difficulties during construction, particularly during pulverizing and blending. Soils which contain appreciable amounts of organic matter are not suitable for soil-cement stabilization.

68. Materials

a. Cement. The cement used is standard portland cement.

b. Water. Any normal source of water may be used, although it should be free from excessive amounts of organic materials, acids, and alkalies.

c. Soil. The maximum size of aggregates used in soil-cement is 3 inches, and the soil should not contain more than about 50 percent of material which is retained on a No. 4 sieve. Experience

in highway construction indicates that, for successful and economical soil-cement stabilization, the soil should not contain more than 50 percent material passing a No. 200 sieve, should have a liquid limit less than 40, and a plasticity index less than 18. An important factor in determining the suitability of a given soil for soil-cement construction is the ease with which the soil can be pulverized.

d. Soil-Cement Mixture. The amount of cement which is necessary for use with a given soil for satisfactory and economical results cannot be determined by a simple procedure. Accurate determination of the proper cement content is based upon a rather elaborate series of laboratory tests, including the determination of volume changes cauesd by alternate freezing and thawing, and alternate wetting and drying. Such testing is beyond the normal range of facilities available to the military engineer in the field and will not be presented here. As previously indicated, most soils which are suitable for soil-cement can be adequately hardened by the use of from 7 to 14 percent cement by volume of the compacted mixture. Granular soil which contains little binder and is nonplastic or feebly plastic in nature can generally be stabilized by cement contents in the lower end of the indicated range, as can many clean sands. Silts and the more plastic soils require higher cement contents. In field situations in which economy is not the principal factor involved, the engineer should be certain that enough cement is used to insure hardening of the mixture. Laboratory determinations of the optimum moisture content and maximum density of the soil-cement mixture are necessary in order that compaction may be adequately controlled. The optimum moisture content of soil-cement will generally be from 2 to 3 percent greater than that of the untreated soil. Enough water must be present to insure hydration of the cement.

69. Construction Procedures

Soil-cement bases have been most frequently constructed with a compacted thickness of 6 inches. Assuming that the subgrade has been brought to the desired condition, principal steps in the construction process include pulverization of the existing soil, spreading of cement, dry mixing of the soil and cement, addition of water and mixing, compaction and finishing, and curing.

a. Pulverizing Existing Soil. The existing soil is scarified to the depth of processing, usually by the use of a rooter or a scarifier attachment on a blade grader. The soil is then pulverized by the use of disk harrows, plows, cultivators, or rotary tillers (par. 45). It is generally desirable that the soil, with the exception of gravel or stone, be pulverized until at least 80 percent will pass a No. 4 sieve. The moisture content of soils which contain considerable

silt or clay may have to be controlled in order to facilitate pulverization.

b. Spreading of Cement. The proper amount of cement may be spread upon the surface, either by hand or by mechanical means. If cement is in sacks, these are generally spotted along the surface in rows of predetermined spacing. The number of sacks, by volume of compacted admixture, which is required may be calculated as follows: Assume that 10 percent of cement is to be used. The soil-cement layer is to be 6 inches thick, and the width of the processed strip is 24 feet. The volume of cement required per 100-foot station is then 24 (100) (6/12) 0.10 = 120 cubic feet or 120 sacks of cement, since the volume of 1 sack of cement may be taken to be 1 cubic foot. The sacks are then opened and the cement spread in uniform transverse rows. Spreading may be completed by use of a spike-toothed harrow or similar tool.

c. Dry Mixing of Soil and Cement. The soil and cement may be mixed by the use of the tools listed in *a* above. Careful control is necessary during this operation, as in fact throughout the entire procedure, in order that the mixing will be carried to the proper depth and a uniform mixture secured. Available equipment is generally operated in a mixing train, with units following one another in close sequence. Additional units are added to the train to apply water and accomplish wet mixing.

d. Addition of Water and Wet Mixing. After the dry materials have been thoroughly mixed, water is added by a distributor to bring the soil to the proper moisture content for compaction. Mixing is continued until the soil, cement, and water are thoroughly blended. Speed is essential in this phase of the process, because the remaining steps must be completed before the cement hydrates and the mixture hardens. Several passes of the mixing train will generally be required to obtain a uniform mixture.

e. Compaction and Finishing. Initial compaction of a soil-cement base is usually done by sheepsfoot rollers, although pneumatic-tired rollers may be necessary in very sandy soils. After rolling is completed, the surface is brought to final shape. Final shaping may be done with the aid of a spike-toothed harrow or a nail drag, to remove the compaction planes left by the rollers, followed by a broom drag. A pneumatic-tired roller may then be used again, the surface shaped by a blade grader, and compaction planes again removed by a broom drag. Final compaction is then achieved by rollers with steel wheels. Rolling is continued until a tightly packed surface is secured. It is essential that the moisture content be very close to optimum during all the rolling operations which have been described. Frequent checks of moisture and density are usually necessary.

f. Curing. The water which is contained in a soil-cement mixture is necessary to the hardening of the cement and steps must be taken to prevent the loss of moisture from the completed base by evaporation. A protective covering of from 2 to 4 inches of soil may be placed over the base, or hay or straw may be used. A light application of bituminous material may be used for the same purpose; if the base must be opened to traffic immediately after construction, this is the best solution. In some cases it will probably be necessary to make a light application of fine aggregate to prevent pickup of the bituminous material. It is desirable that curing be continued for 7 days before the base is opened to traffic, but in an emergency it can be opened to traffic immediately.

70. Soil-Lime Stabilization

a. Hydrated lime may be used to reduce the plasticity and improve the mixing, compaction, and strength characteristics of some plastic soils. The amount of lime used is quite small, generally being from 2 to 10 percent by weight.

b. Experience has shown that for the more plastic soils (plasticity index more than 15), the plasticity index is reduced by the addition of hydrated lime. The soil is thus easier to pulverize and easier to mix.

c. Under a given compactive effort, the density of a lime-treated soil is less than that of the raw soil. However, the soil may be easier to compact and densities obtained in the field more uniform.

d. However, the strength of compacted lime-soil mixtures is greater than that of similar raw soils. The increase in soil strength is brought about principally by changes in water films surrounding the clay particles. There is some evidence that the strength increases with curing and age.

e. In general, lime-soil mixtures are susceptible to frost actior and deteriorate rapidly under alternate freezing and thawing A typical mixture of this type has poor weathering characteristics.

f. The construction of a lime-soil subgrade or base should present no particular difficulties, although the lime must be thoroughly mixed with the soil. It is generally spread on the soil layer and mixed in the same fashion as soil-cement. The moisture content must be brought to optimum and the mixture compacted in the same way as the natural soil. Time is not a critical factor from the standpoint of the reaction between the lime and the soil, since the lime will not harden within the normal construction day.

g. Recently the Waterways Experiment Station has conducted extensive tests, using quick instead of hydrated lime, with promising results.

CHAPTER 8

BASE COURSES

71. Essential Features

The purpose of a base course is to distribute the induced stresses from the wheel load so that they will not exceed the strength of the subgrade. Figure 81 shows an idealized representation of the distribution of stress through two base courses. When the subgrade strength is low, the stress must be reduced to a low value and a substantial thickness of base is needed. Where the subgrade strength is higher, a lesser thickness will provide adequate distribution. Since the stresses in the base course are always higher than in the subgrade (fig. 81) it stands to reason that the base course must have a high strength. Similarly, where two or more different types of base courses are used, the better quality material is placed on top. Required thickness of base may be reduced by building higher strength into the subgrade.

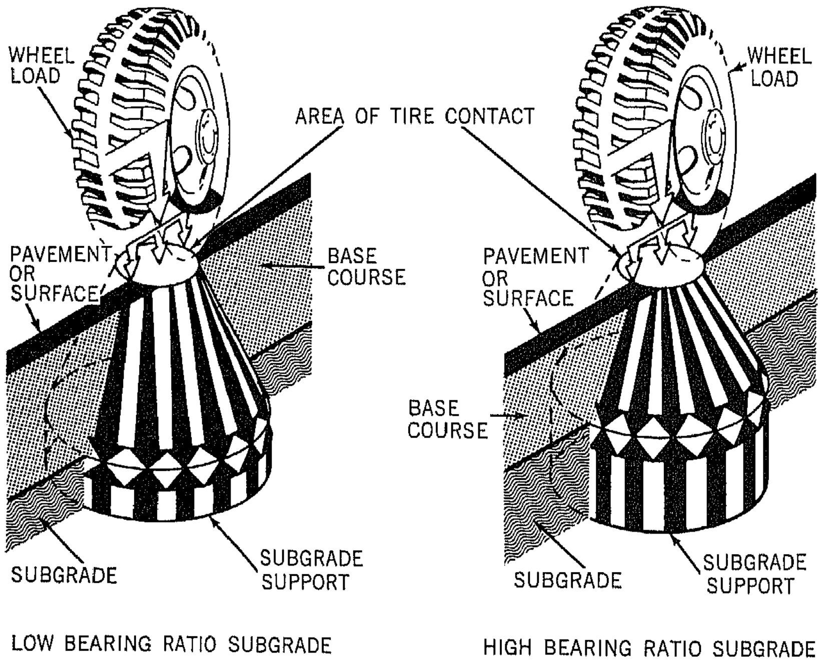


Figure 81. Distribution of stress in base courses and effects of subgrade strength on base-course thickness.

72. Base-Course Requirements

Careful attention should be given to the selection of materials for base courses and to their construction. The materials should be dense and uniformly compacted so no differential settlement occurs in adjacent areas. For continuous stability, all base courses should meet the requirements listed below.

a. Gradation Requirements.

(1) Gradation of particle size must, whenever feasible, be within specified limits as determined by mechanical analysis. However, in construction in forward areas it many not be practicable to hold to close gradation requirements. Deviation from gradation requirements should be permitted only if all other characteristics, especially the California Bearing Ratio, are satisfactory.

(2) Base-course mixtures must be uniform as to grain-size distribution over the length and width of the base course, with no accumulations in local areas of either coarse or fine particles, especially material passing the No. 40 sieve.

(3) For deliberate construction base course material should contain no more than 15 percent passing a No. 200 sieve.

b. Plasticity Requirements. Material passing the No. 40 sieve which acts as a binder in a base-course material must have desirable water-resistant qualities, as shown by satisfactory values of the liquid limit and plasticity index. Requirements for mechanically stabilized soil bases are given in paragraph 56. No material which has a liquid limit greater than 25 or a plasticity index greater than 5 should be used for a base course in deliberate construction.

c. Compaction and Strength Requirements.

(1) Thickness of layers in constructing base courses must be within the limits which will insure proper compaction. Thickness of layers depends upon type of material, equipment used, and method of construction used.

(2) All base courses must be compacted. Compaction should meet the requirements given in paragraph 47.

(3) The bearing ratio of the finished base must conform to that used in the design, and the total compacted thickness must equal that obtained from the design curves, as discussed in TM 5-330.

73. Base-Course Materials

a. Natural Materials. A wide variety of gravels, sands, gravelly and sandy soils, and other natural materials such as limerock, coral, shells, and some caliches, can be used alone or blended to

provide satisfactory base courses. In some instances, natural materials will require crushing or removal of the oversize fraction to maintain gradation limits. Other natural materials may be controlled by mixing crushed and pit-run materials to form a satisfactory base-course material.

(1) *Gravel and sand.* Many natural deposits of sandy and gravelly materials make satisfactory base materials. Gravel deposits vary widely in the relative proportions of coarse and fine material and in the character of the rock fragments. Satisfactory base materials often can be produced by blending materials from two or more deposits. Uncrushed clean washed gravel is not satisfactory for a base course, because the fine material which acts as the binder and fills the void between coarser aggregate has been washed away. A base course made from sandy and gravelly material which meets the requirements given in paragraph 72 has a high bearing value and can be used to support heavy loads.

(2) *Sand-clay.* Sand and clay in a natural mixture may be found in alluvial deposits varying in thickness from 1 to 20 feet. Often there are great variations in the proportions of sand and clay from top to bottom of a pit. Deposits of partially disintegrated rock consisting of fragments of rock, clay, and mica flakes should not be confused with sand-clay soil. Mistaking such material for sand-clay is often a cause of base-course failure because of reduced stability due to the mica content. With proper proportioning (par. 58) and construction methods, satisfactory results can be obtained with sand-clay. It is excellent in stage construction where a higher type of surface is to be added later. Requirements for sand-clay base courses are given in paragraph 56.

b. Stabilized Soil Mixtures. The stabilization of subgrade soils by various methods is discussed in detail in paragraphs 51 through 70. All three principal types of stabilized soil mixtures can be used as base courses beneath bituminous wearing surfaces if tire pressures are less than 100 psi. Their use as base courses is most common on roads which carry light to moderate traffic, although they have also been used on airfields in expedient situations.

c. Processed Materials. Processed materials are prepared by crushing and screening rock, gravel, or slag. A properly graded crushed-rock base produced from sound, durable rock particles makes the highest quality of any base material. Crushed rock may be produced from almost any type of rock that is hard enough to require drilling, blasting, and crushing. Existing quarries, ledge

rock, cobbles and gravel, talus deposits, coarse mine tailings, and similar hard, durable, rock fragments are the usual sources of processed materials. Materials which slake on exposure to air or water should not be used. Processed materials should not be used if materials such as gravel or sand-clay are available, except when studies show that the use of processed materials will save time and effort or when they are made necessary by project requirements. Bases made from processed materials can be divided into three general types: composite, coarse graded, and macadam.

(1) *Composite type base material.* A composite type base course is one in which all the material ranging from coarse to fine is intimately mixed either before or as the material is laid into place. If practicable, materials for this type of base should meet the requirements given in paragraph 56 and table XV. Because the aggregates produced in crushing operations or obtained from deposits are often deficient in fines, it may be necessary to blend in selected fines to obtain a suitable gradation (par. 58). Screenings, crusher-run fines, or natural soil containing no clay may be added and mixed either in the processing plant or during the placing operation.

(2) *Coarse-graded type base material.* A coarse-graded type base course is composed of well-graded crushed rock, gravel, or slag. If practicable, materials for this type of base should meet the gradation requirements given in table XVI. This base may be used to advantage when it is necessary to produce crushed rock, gravel, or slag on the site or when commercial aggregates are available. When gravel is used, it is desirable that 90 percent of the material by weight have two or more freshly fractured faces, with the area of each face being equal to at least 75 percent of the smallest midsectional area of the piece.

(3) *Macadam type base material.* The term macadam is usually applied to construction where a coarse, crushed aggregate is placed in a relatively thin layer and rolled into place; then fine aggregate or screenings are placed on the surface of the coarse aggregate layer and rolled and broomed into the coarse rock until it is thoroughly keyed in place. Water may be used in the compacting and keying process. When water is used, the base is termed a waterbound macadam. The crushed rock used for macadam base courses should consist of clean, angular, durable particles free of clay, organic matter, and other objectionable material or coating. Because of

Table XVI. Desirable Gradation for Crushed Rock Types of Crushed Rock, Gravel, pr Slag Aggregates for Base Courses

Sieve designation	Percent passing each sieve (square openings) by weight — Maximum aggregate size: 3-inch	2-inch	1½-inch	1-inch
3-inch	100			
2-inch		100		
1½-inch		70–100	100	
1-inch	35–65	45–80		100
½-inch		30–60		
No. 4	10–30	30–50	20–40	25–45
No. 10		15–40		
No. 40		5–25		
No. 200		(*)	(*)	(*)

*Less than 10.

*Table XVII. Desirable Gradations of Aggregate for Drybound and Waterbound Macadam Base Courses**

(from CE Guide Specifications CE 807.05 and CE 807.06 dated 15 September 1959)

DRYBOUND

Sieve designations	Percentage by weight passing each square-mesh sieve: Coarse aggregate gradation				Choker aggregate gradation		Screenings
	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7
3-inch	100	100					
2½-inch	90–100	90–100	100				
2-inch	35–70		90–100	100			
1½-inch	0–15	25–60	35–70	90–100			
1-inch			0–15	20–55			
¾-inch	0–5	0–10		0–15	100		
½-inch		0–5	0–5		90–100		
⅜-inch				0–5		100	100
No. 4						85–100	85–100
No. 100					10–30	10–30	5–25

WATERBOUND

Sieve designations	Percentage by weight passing each square-mesh sieve: Coarse aggregate gradation			Screening gradation	
	No. 1	No. 2	No. 3	No. 4	No. 5
3-inch	100	100			
2½-inch	90–100	90–100	100		
2-inch	35–70		90–100		
1½-inch	0–15	25–60	35–70		
1-inch			0–15		
¾-inch	0–5	0–10		100	
½-inch		0–5	0–5	90–100	
⅜-inch					100
No. 4					85–100
No. 100				10–30	10–30

*The portion of aggregate passing the No. 40 sieve shall be either nonplastic or shall have a liquid limit > 25 and a plastic limit < 5.

the method of construction, it is necessary to maintain the coarse and fine aggregates separately. Aggregates for macadam type construction should meet the gradation requirements given in table XVII. Any hard, durable crushed aggregate can be used, provided the coarse aggregate is primarily one size and the fine aggregate will key into the coarse aggregate.

d. Other Materials. In many areas in a theater of operations, deposits of natural sand and gravel and sources of crushed rock are not available. This has led to the development of base courses from materials that normally would not be considered. These include caliche, limerock, shells, cinders, coral, iron ore, rubble, and other select materials. Some of these are primarily soft rock, and crush or degrade under construction traffic to produce composite base materials similar to those described in *c*(1) above. Others develop a cementing action which results in a satisfactory base. These materials cannot be judged on the basis of the gradation limits used for other materials, but must be judged on the basis of service behavior. Strength tests on laboratory samples are not satisfactory because the method of preparing the sample seldom duplicates the material in place. The plasticity index is a reasonably good criterion, and as a general rule a low plasticity index is a necessity. Observation of the performance of these types of base materials in existing roads and pavements is the most reliable clue as to whether or not they will be satisfactory.

(1) *Coral.* Coral is commonly found in the Pacific and Caribbean areas. Since uncompacted coral has a high capillarity, uncompacted and poorly drained coral often results in an excessive moisture content and loss of stability. The bonding properties of coral, which are its greatest asset as a construction material, vary with the amount of volcanic impurities, the proportion of fine and coarse material, age, length of exposure to the elements, climate, traffic, sprinkling, and method of compaction. Proper moisture control, drainage, and compaction are essential to obtain satisfactory results. Sprinkling with sea water or sodium chloride in solution has a beneficial effect on bonding when rollers are used. However, because of the possible corrosive effects on aircraft engines, sprinkling with sea water on airfields should be undertaken only with specific approval of higher authority.

(2) *Caliche.* One of the fairly common characteristics of many caliches which make them valuable for base courses is their quality of recementation upon being saturated by water, subjected to compaction, and given a setting period. This is especially applicable to caliches which

are cemented with lime, iron oxide, or salt. Caliche is variable, however, in content (limestone, silt, and clay) and in the degree of cementation; therefore, it is important that caliche of good uniform quality be obtained from deposits, and that it be compacted at optimum moisture. After caliches have been slaked for 72 hours the liquid limit of the fines should not exceed 35, and the plasticity index should not exceed 10. For base course material, caliches should be crushed to meet the following gradations:

	Percent
Passing 2-inch square mesh	100
Passing No. 40 sieve	15–35
Passing No. 200 sieve	0–20

Where the construction is to be made on surface deposits, undesirable material is removed by stripping operations.

(3) *Tuff.* Tuff and other cementaceous materials of volcanic origin may be used for base courses. Tuff bases are constructed the same as other base courses except that after the tuff is dumped and spread, the oversize pieces are broken and the base compacted with sheepsfoot rollers. The surface is then graded, and final compaction and finishing are done.

(3) *Rubble.* In many cases it may be advantageous to use the debris or rubble of destroyed buildings in constructing base courses, but care must be exercised to see that jagged pieces of metal and similar objects are removed. Before removing any rubble, a check should be made for mines or boobytraps.

e. Bituminous Base. Bituminous plant mixtures frequently are used as base courses beneath high type bituminous pavements, particularly for permanent type airfields which carry heavy traffic. Such base courses may be used to advantage when aggregates locally avalable are relatively soft and otherwise of relatively poor quality, when mixing plant and bituminous materials are readily available, and when a relatively thick structure is required for the traffic. In general, a bituminous base course may be considered equal on an inch-for-inch basis to other types of high-quality base courses. When a bituminous base course is used, it will be placed in lifts not exceeding 3½ inches in thickness. If a bituminous base is used, the binder course may be omitted and the surface course may be laid directly on the base course.

74. Selection of Type of Base Course

Selection of the type of base construction depends principally upon materials available at the particular site, but equipment available and prevailing weather conditions during construction

also are important factors. A complete investigation should be made to determine the location and characteristics of all natural materials suitable for base-course construction. Construction of untreated base courses with natural materials is affected less by adverse weather than other types and requires less technical control. Untreated bases are relatively easy and fast to build and are recommended in preference to bituminous- or cement-stabilized types, except where suitable materials for such construction are more readily available. If they are not locally available, the transportation of bituminous material or cement for base stabilization is a major supply problem in forward areas

75. Construction Operations

a. Job Organization. Operations are organized in accordance with the plan and schedule set up for the job.

b. Fine Grading. The subgrade is fine graded where necessary with a motorized or towed type grader to bring it to the designed cross section established by final grade stakes. Any ruts or soft, yielding areas that appear in the subgrade by reason of improper drainage conditions, hauling, or other causes, should be corrected and rolled until firm before the base-course material is placed.

c. Spreading Forms and Stakes. Spreading forms and stakes are seldom used on combat roads in placing base-course materials. They should be used on roads built in rear areas and designed for heavy traffic. When roads are constructed by the trench method the berms formed by the shoulders serve as forms. In airfield construction, spreading stakes are generally used as guides in spreading loose base material to uniform thickness. They should be spaced on 20- to 30-foot centers both across and along the runway and driven so that their tops are at the desired height of the loose spread. In some cases spreading forms are practical to use in airfield construction. Forms made of random length lumber, 2 to 3 inches thick, are preferred. Their height equals the loose-layer thickness of the material to be spread. The forms are laid on edge along the sides of the runway and on intermediate parallel lines 20 to 30 feet apart. They are held in place by brackets nailed to the sides, by stakes along the edges, or by a few shovelfuls of base material banked along each side. As soon as spreading is completed on one section, the forms are pulled up and moved ahead. Field-trial or laboratory compaction tests determine the depth of loose spread required to produce compacted thickness. When specific data is lacking, it is assumed loose thickness will be 1.25 times the compacted thickness.

d. Hauling, Placing, and Spreading.

(1) The placing and spreading of the material on the prepared subgrade, or on a completed layer, may begin at

the point nearest the source or at the point farthest from the source; material is then placed progressively away from or toward the source, respectively. The advantage of working from the point nearest the source is that the hauling vehicles can be routed over the spread material, which assists in compacting the base and avoids cutting up the subgrade. Advantages of working from the point farthest from the source are that hauling equipment will further compact the subgrade, reveal any weak spots in the subgrade so that they can be corrected promptly, and interfere less with the movement of spreading and compacting equipment. When hauling vehicles are not desired on the subgrade and placing begins at the point farthest from the source, hauling vehicles should be routed over adjacent finished working strips and the material spread transversely at the point of deposit. In this case, working strips on runways are usually 500 to 1,500 feet long, and the entire width of the runway is completed for each working strip length so that surfacing can begin as soon as practicable.

(2) Trucks, scrapers, or other hauling vehicles deposit material directly on the subgrade. Proper dumping methods should be used to minimize the amount of spreading required. Towed aggregate spreaders are sometimes used, or tailgate openings are restricted to control the rate of spread from trucks. Spreading is accomplished with a dozer or grader. The dozer blade is set straight and at right angles to the direction of laying, and the dozer runs on the base at all times. The forward motion of the dozer spreads the material, and the desired leveling and smoothing are obtained by dragging the blade when backing up. Care should be taken to avoid segregation of the material.

(3) The base-course material should be transported from the plant or stockpile to the prepared subgrade or subbase in pneumatic-tired vehicles to prevent possible damage to the prepared subgrade. The vehicles should have dump bodies suitable for dumping materials in windrows or into spreading machines. The mixed material should be placed on the prepared subgrade or subbase in layers of uniform thickness with an approved mechanical spreader. When a compacted layer 6 inches thick or less is required, the material should be placed in a single layer; when a compacted thickness in excess of 6 inches is required it shall be placed in two or more layers of equal thickness, but not to exceed 6 inches. The layers

shall be placed so that when compacted they will be true to the grades or levels required with the least possible surface disturbance. The water content of the material should be maintained at the percentage specified during the placing period. Adjustments in placing procedures or equipment should be made, if necessary, to obtain true grades, to minimize segregation and creation of fines, to reduce or accelerate loss or accretion of water, and to insure a satisfactory base course.

e. Blending and Mixing. Materials to be blended and mixed on the road, runway, or taxiway are spread evenly in correct proportions with the finer material on top. Initial mixing to work the fine into the coarse material may be accomplished with the scarifier attachment or with harrows. Final mixing is accomplished with a rotary tiller or a grader. When a grader is used, the materials are thoroughly mixed by alternately blading the entire layer to the center and back to the edges of the working strip. The coarse and fine aggregates can also be mixed in approved mechanical plants of the traveling or stationary type or on a paved area with blade graders and bucket loaders. The coarse and fine aggregates should be proportioned by weight or by volume in quantities such that the specified gradation, liquid-limit, and plasticity-index requirements will be met after the base has been placed and compacted. Mixing operations should produce uniform blending. Placing into and dumping from trucks should not produce segregation. When mechanical mixing is used, the coarse and fine aggregates should be placed in separate stockpiles or adjacent windrows to permit easy proportioning. When bucket loaders are used, the fine and coarse aggregate portions should be placed in adjacent windrows on a paved area, after which the windrows should be bladed together to meet the requirements specified for the mixing plants above.

f. Watering Base Materials. As in the subgrade-compaction operations, the obtaining of the specified compacted density will generally entail use of a moisture content slightly greater or less than the optimum during mixing, depending on the nature of the material and/or the atmospheric conditions. The addition of the correct quantity of water during mixing to obtain the required moisture content is again dependent on the experience and judgment of the engineer. All laboratory data on moisture should be made available to the engineer in charge. Base material is maintained at optimum moisture content by watering with a truck-mounted tank through the trailer-mounted water distributor. Various field expedients may be used, but asphalt distributors are not used except in extreme emergencies because the pump-lubrication system is not designed for water distribution. The haul road

can sometimes be routed past a waterhole where the material can be watered in the hauling equipment, incurring a delay of only 1 or 2 minutes a load.

g. Compacting.

(1) The base-course compaction operations must produce a uniformly dense layer conforming in every way to specification requirements. Each of the base-course materials should be compacted with heavy rubber-tired rollers. Water content should be maintained at saturation during the compaction procedure or at the percentage specified. In all places not accessible to the rollers, the mixture should be compacted with mechanical tampers. Compaction should continue until each layer is compacted through the full depth to the specified percentage of either the standard or modified AASHO compaction test. Field densities should be measured on the total sample. The engineer should make such adjustments in rolling or finishing procedures as may be necessary to obtain true grades. The care and judgment used in the construction of the base course will be reflected directly in the quality of the finished flexible pavement.

(2) Base-course layers containing gravel and soil-binder material may be compacted initially with a sheepsfoot roller and 13-wheel, rubber-tired roller. Heavy self-propelled rubber-tired rollers are also very satisfactory for compacting this type of material. Rubber-tired rollers are particularly effective in compacting base materials if a kneading motion is needed to adjust and pack the particles. Base courses of crushed rock, limerock, and shell are compacted with rubber-tired rollers or with 3-wheel steel rollers. Equipment and methods must be adjusted on each job to suit the characteristics of the base material (fig. 61), since thorough compaction is highly important in developing maximum stability.

h. Finishing. Finishing operations should closely follow compaction to furnish a crowned, tight, water-shedding surface free of sheepsfoot-roller holes which prevent runoff. The motor grader is used for finishing graded aggregate bases. The material is bladed from one side of the runway, taxiway, or road, to the middle and back to the edge until the required lines and grades are obtained. Before final rolling, the bladed material must be at its optimum moisture content, so that it will consolidate with the underlying material to form a dense, unyielding mass. If this is not done, thin layers of the material will not be bound to the base and peeling or scabbing may result. Final rolling is done with 13-wheel, rubber-tired rollers and 3-wheel steel rollers.

i. Special Procedures for Macadam Bases.

(1) *Subgrade preparation.* If the subgrade soil is plastic and cohesive, a 1- to 2-inch layer of sand or stone screenings should be spread, sprinkled, and rolled onto the subgrade before spreading the coarse macadam aggregate, as described in paragraph 73*c*(3).

(2) *Spreading.* Care must be exercised in placing and spreading macadam aggregate to insure that hauling vehicles do not add dirt or other objectionable material to the aggregate. This care is particularly necessary when placing from the point nearest to the source and routing hauling vehicles over the spread material. If the compacted thickness is to be 4 inches or less, the loose macadam aggregate is spread in a uniform layer of sufficient depth to meet requirements. For greater compacted thickness, the aggregate is applied successively in two or more layers. Spreading should be from dump boards or towed aggregate spreaders, or from moving vehicles controlled to distribute in a uniform layer. When more than one layer is required, construction procedure is identical for all layers.

(3) *Compaction.* Immediately following spreading, the coarse aggregate is compacted the full width of the strip by rolling with the 10-ton, 3-wheel steel roller. Rolling progresses gradually from the sides to the middle of each strip in a crown section, and from the low side to the high side where there is a transverse slope completely across the road, runway, or taxiway. Rolling continues until the absence of creep or wave movement of the aggregate ahead of the roller indicates that the aggregate is thoroughly keyed. Rolling is not attempted when the subgrade is softened by rain. After rolling, the finished surface should not have irregularities that exceed $\frac{3}{4}$ inch when tested with a 10-foot straightedge. Irregularities should not exceed $\frac{3}{8}$ inch in the case of runways for jet aircraft.

(4) *Applying screenings.* After the coarse aggregate has been thoroughly keyed and set by rolling, sufficient screenings are distributed to fill the voids in the surface. Rolling is continuous while screenings are being spread, so the jarring effect of the roller will cause them to settle into surface voids. Screenings are spread in thin layers by hand shovels or mechanical spreaders or directly from moving trucks. They are not dumped in piles on the coarse aggregate. If necessary, hand or drag brooms may be used to distribute screenings during rolling. Care

must be exercised to see that screenings are not applied too fast or thick, or they will cake or bridge on the surface and prevent the filling of some of the voids, or prevent the direct bearing of the roller on the coarse aggregate. Spreading, sweeping, and rolling continue until no more screenings can be forced into the voids, but excess of screenings is avoided. The finished dry macadam base must be true to line and grade. In waterbound-macadam construction, sprinkling and grouting start after screenings have been spread. A section of convenient length is sprinkled until saturated, then rolled. Care is taken to prevent saturation and softening of the subgrade. Surface screenings are added as needed. Sprinkling and rolling continue until a grout of screenings and water forms, fills all voids, and gathers in a wave before each roller. Hand or power brooms are used to sweep wet screenings into unfilled voids. When a section of a strip has been grouted thoroughly, it is allowed to dry completely before additional work is done on it.

76. Finished Surface

a. Relation to Finished Pavement. The nature of the finished base-course surface is one of the determining factors in the smoothness of the surface of the finished pavement. It must be recognized that if the finished base does not conform to the specified grade when tested with a straightedege, the finished pavement probably will not do so. The surface of the base should be reasonably smooth and aggregate faces should be showing. The surface should conform to specified requirements.

b. Smoothness Test. It is recommended that the surface of the base course should not show any deviation in excess of $\frac{3}{4}$ inch for roads and airfields for propeller-type planes, or $\frac{3}{8}$ inch for jet airplanes when tested with a 10-foot straight edge applied both parallel with and at right angles to the center line of the paved area. Any deviation in excess of this amount should be corrected by removing material to the total depth of the lift and replacing with new material, and compacting as specified above.

c. Slush-Rolling.

(1) *Use.* The ultimate purpose of slush-rolling (rolling with enough water to produce a slushy surface) is merely to smooth and tighten the surface rather than to secure additional compaction of the base course. Slush rolling should be permitted on a cured base course only. If the surface is generally satisfactory but there are some relatively large areas which require slushing, the entire area of base course should not be slushed; this treatment

should be confined to the rough areas alone. Slushing
makes fines and brings
sive layer or fines is st
duce adequate bond. It
slush rolling should not
course material and sh
by the specification

(2) *Water application*
but only on the su
greatly with the
temperature, and
starting is 0.5 to
must follow im
achieve the des
carry a wave of
base course. If t
base, the entire b
with subsequent
gate interlock. T
truck and the ro

(3) *Rollers.* Three-
should be used.
the desired surface has been

(4) *Finishing.* After slushing is completed there
erally small rivulets or ridges of fines left on the surface. Where these are excessive, or where the thickness of the blanket of fines is excessive, the surface should be sprinkled and honed off (dressed lightly) with a blade (motor grader). This is a delicate operation requiring a good operator and a sharp, true blade. The blade should be followed immediately with a pneumatic roller to reset the surface.

d. Wet-Rolling.

(1) *Use.* All base courses require a final finish of the surface, but the high-type base courses, with their improved requirements for gradation, liquid limit, and plasticity index, should obtain such a finish automatically after the final compaction and/or proof-rolling. For less critical base courses, or where deemed necessary by the construction agency, wet rolling as well as slush-rolling may be used to obtain a final finish of the surface. Each method has its strong points and in some cases the particular job may require a combination of the two.

(2) *Water application.* Wet rolling does not require the large amount of water demanded for slush rolling and the base

course need not have gone through the curing period required by the slush-rolling method. The water is applied to the base course in the amount that will raise the moisture content of the upper 1 to 2 inches of the base course to approximately 2 percentage points above the optimum moisture content. The percent moisture will vary with the type of material and is a matter of judgment by the field inspection forces.

(3) *Finishing.* Finishing the surface is accomplished by having the blade make a light cut followed immediately by the pneumatic-tired roller and the 10-ton, steel-wheeled roller. The light blading will loosen the fines; the coarser particles of the base course will be carried along by the blade to form a windrow at the edge of the section being finished. This coarse aggregate can be evenly distributed over the area and rear-wheeled into the surface of the base by the 10-ton, steel-wheeled roller as described previously. Additional water may be required and the rolling by the steel-wheeled and pneumatic-tired rollers must be continued until a smooth, dense surface is obtained. This method can also be used for correcting minor surface irregularities in the base course.

77. Technical Control

Although important, visual inspection alone is not sufficient for control of the construction of the bases described in this chapter, particularly for those which contain considerable fine material. Depending on the type of base, necessary control tests will include determinations of gradation, mixture proportions, plasticity characteristics, moisture content, field density, thickness, and CBR values. These tests are described in detail in TM 5–530. Construction operations must be altered if requirements are not being met.

APPENDIX
REFERENCES

1. DA Pamphlets (DA Pam).

DA Pam 108–1	Index of Army Motion Pictures, Film Strips, Slides, and Phono-Recordings.
DA Pam 310-series	Military Publications Indexes.

2. Army Regulations (AR).

AR 320–50	Authorized Abbreviations and Brevity Codes.
AR 320–5	Dictionary of United States Army Terms.

3. Field Manuals (FM).

FM 5–34	Engineer Field Data.
FM 5–35	Engineer Reference and Logistical Data.
FM 5–36	Route Reconnaissance and Classification.
FM 30–10	Terrain Intelligence.

4. Technical Manuals (TM).

TM 5–233	Construction Surveying.
TM 5–252	Use of Road and Airfield Construction Equipment.
TM 5–258	Pile Construction.
TM 5–260	Principles of Bridging.
TM 5–302	Construction in the Theater of Operations.
TM 5–330 (To be published)	Planning, Site Selection, and Design of Theater of Operations Roads, Airfield and Heliports.
TM 5–332	Pits and Quarries.
TM 5–337	Bituminous, Concrete, and Expedient Paving Operations.
TM 5–530	Materials Testing.
TM 5–541	Control of Soils in Military Construction.

INDEX

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NG: State AG (3); units—same as Active Army except allowance is one copy to each unit.

USAR: Same as Active Army except allowance is one copy to each unit.

For explanation of abbreviations used, see AR 320-50.

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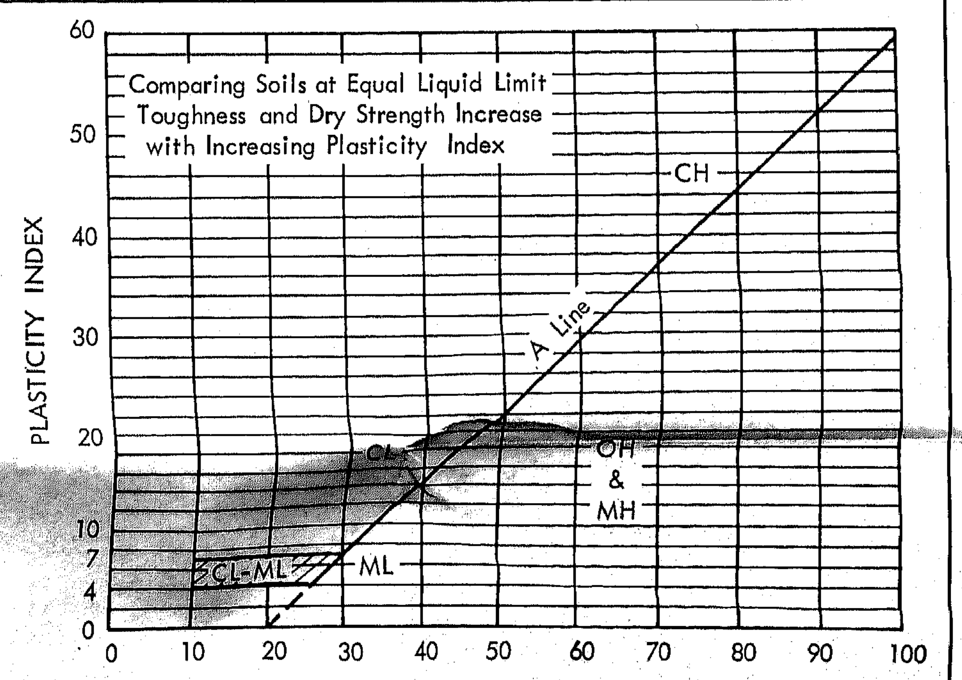
UNIFIED SOIL CLASSIFICATION
(Including Identification and Description)

Major Divisions (1)	Major Divisions (2)		Group Symbols (3)	Typical Names (4)	Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights.) (5)	Information Required for Describing Soil (6)	Laboratory Classification Criteria (7)
Coarse-grained Soils. More than half of material is larger than No. 200 sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels. More than half of coarse fraction is larger than No. 4 sieve size. (For visual classification, the 1/4-in. size may be used as equivalent to the No. 4 sieve size)	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics. Give typical name; indicate approximate percentage of sand and gravel, maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses. Example: Silty sand, gravelly; about 20% hard, angular gravel particles 1/2-in. maximum size; rounded and subangular sand grains, coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).	Use grain-size curve in identifying the Fractions as given under field identification. Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5% GW, GP, SW, SP. More than 12% GM, GC, SM, SC. 5% to 12% Borderline cases requiring use of dual symbols. — $C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		Not meeting all graduation requirements for GW
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixture.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below)		Atterberg limits below "A" line or PI less than 4. Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below.)		Atterberg limits above "A" line with PI greater than 7
	Sands. More than half of coarse fraction is smaller than No. 4 sieve size	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	Wide range in grain size and substantial amounts of all intermediate particle sizes.		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			SP	Poorly graded sands or gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		Not meeting all graduation requirements for SW
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below)		Atterberg limits below "A" line or PI less than 4. Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below.)		Atterberg limits above "A" line with PI greater than 7

Major Divisions (1)	Major Divisions (2)	Group Symbols (3)	Typical Names (4)	Identification Procedures on Fraction Smaller than No. 40 Sieve Size: Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)	Information Required for Describing Soil (6)
Fine-grained Soils. More than half of material is smaller than No. 200 sieve size.	Silts and Clays. Liquid limit is less than 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None	For undisturbed soils add information on structure stratification, consistency in undisturbed and remolded states, moisture and drainage conditions. Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name and other pertinent descriptive information; and symbol in parentheses. Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium	
		OL	Organic silts and organic silts clays of low plasticity.	Slight to Medium	Slow	Slight	
	Silts and Clays. Liquid limit is greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium	
		CH	Inorganic clays of high plasticity, fat clays	High to very high	None	High	
		OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to mdeium	
Highly Organic Soils		Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.			



PLASTICITY CHART

For laboratory classification of fine-grained soils

(1) Boundary classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For examples GW-GC, well-graded gravel-sand mixture with clay binder. (2) All sieve sizes on this chart are U.S. standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air-drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt posses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (consistency near plastic limit)

After particles larger than the No. 40 sieve size are removed, a specimen of soil about one-half inch in cube size, is [illegible] to the consistency of putty. If too dry, water must be added and if sti[illegible]
thin layer and allowed to lose some
is rolled out by hand on a smooth su[illegible]
one-eighth inch in diameter. The t[illegible]
During this manipulation the moistur[illegible]
men stiffens, finally loses its plasti[illegible]
is reached.

After the thread crumbles, the pieces
action continued until the lump cru[illegible]

The tougher the thread near the plasti[illegible]
finally crumbles, the more potent is
Weakness of the thread at the plasti[illegible]
lump below the plastic limit indicate
materials such as kaolin-type clays [illegible]
Highly organic clays have a very weak

Figure 59. Unified Soil Classification System.